Pushover Analysis for Earthquake Design of Structures

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Abstract

The Egyptian Code (ECP-201), which includes regulations for earthquake design, was recently finalized and made available for immediate use. The ECP-201 design processes are often based on an elastic force analysis approach. Nonlinear static analysis, often known as pushover analysis, is increasingly being used by seismic engineers because it accurately forecasts where and how much plastic yielding will occur in a given structure. In this article, we used pushover analysis to measure how well these ECP-designed buildings performed. We compare three common types of RC frames. To determine the maximum safe load for a building, a pushover study must be conducted. Building performance is defined in accordance with ATC-40, FEMA-356, and FEMA-440. Based on the results of performance points for the two case studies, the buildings can sustain seismic base shear ranging from 65% to 85% of their ultimate capacity from pushover analysis (POA) in X- and Y-Directions. The findings demonstrate that, on the whole, the acceptance criteria for these methodologies may be applied to buildings designed using ECP.

***Keywords**:* Seismic assessment, Pushover analysis · Nonlinear static analysis, Performance-based evaluation.

1. Introduction

Recently, different countries around the world have been hit by horrific earthquakes with effects ranging from irreparable damage to irreparable damage. As a result, Egypt has paid close attention to understanding how concrete buildings behave during earthquakes.

This research presents through numerical simulation nonlinear static analysis to evaluate the expected performance on a commercial building of reinforced concrete variable in the number of floors consisting of 15, 20, and 25 floors with two basements for each building and the structural system consists of frames that resist moments and is located in the New Administrative Capital. For this purpose, the ETAPS program was used to model the building and analyze it using the non-linear static analysis (PUSHOVER).

At first, the analysis was performed under the influence of equivalent static loads, and then the non-linear static analysis was performed according to what was recommended by international codes and references specialized in seismic assessment such as ATC40. FEMA365 - FEMA 440 - under the influence of two different levels of earthquakes in two perpendicular directions. The results of the study showed that properly designed buildings perform well under the influence of seismic loads that Cairo is exposed to, and it is also clear that the current structure behaves similar to the mechanism of the weak beam - the strong column.

However, under the influence of the unexpected seismic loads assigned to the city of Cairo, the plastic joints appear at a dangerous level according to the classification of the previous codes, and some of them need to be strengthened as a result of the damage. In the event that the design is not sufficient in the shear, the ductile joints will be formed as a result of the shear, and the inelastic failure is dominant in the inelastic behavior.

Recently prepared and made available for immediate application, the Egyptian code (ECP-201, 2020) [1] includes seismic design regulation. The design of buildings and other structures must adhere to the minimum load requirements established in this code. Both the maximum permitted load and the maximum load that may be safely applied are specified.

The ECP design process typically uses a linear force analysis method as opposed to a displacement analysis method. Most structural engineers like the linear analysis method since it is easy to compute and has been around the longest. The method takes into consideration the nonlinear response of structures by means of the response reduction parameter for the entire structure [2, 3].

Various methodologies exist for determining the seismic performance of a building, which vary depending on the specified requirements. FEMA 356 [4] proposes the use of the displacement coefficient method (DCM), while ATC40 [5] provides information on the capacity spectrum technique (CSM). FEMA 440 [6] introduced enhancements to both the DCM (Displacement Control Method) and CSD (Capacity Spectrum Method).The Eurocode 8 [7] implemented the N2 technique, which demonstrates a modified version of the CSM.

Performance-based design criteria that estimate nonlinear response of the building have been utilized in the most up-to-date building standards (e.g., ATC, FEMA) for seismic design and evaluation. Building a model of the structure and running simulations of how it would respond to an anticipated seismic excitation are the first steps in performance-based design. So that a structural engineer can control the risk of damage in terms of recovery cost, each simulation offers the level of damage.

The force-displacement curve in Figure 1 depicts the behavior of the global structure under lateral load and displays the performance level of the building presented. Nonlinear static analysis, often known as pushover analysis, is used to derive the curve.

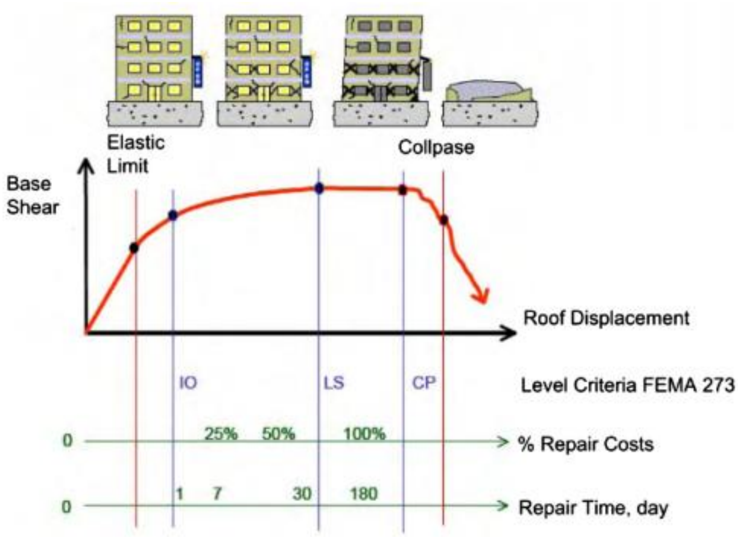


Figure 1. Performance-based design concept

In performance-based design, the engineer designs a structure in response to a specified performance level for the building. Table 1 displays performance levels in accordance with ATC-40 [5].

Table 1. Performance level of buildings

Level	Description
Operational	Very little damage, temporary drift, structure retains original strength and stiffness, all systems are normal
Immediate occupancy	Little damage, temporary drift, structure retains original strength and stiffness, elevator can be restarted, fire protection still works
Life safety	Fair damage, some permanent drift, some residual strength and stiffness left, damage to partition, building may be beyond economical repair
Collapse prevention	Severe damage, large displacement, little residual stiffness and strength but loading bearing column and wall function, building is close to collapse

Multiple studies have elucidated the response of reinforced concrete structures when exposed to seismic activity. Kadid and Boumrkik [8] performed a Performance-based Optimization Analysis (POA) on three buildings with different framing systems, each having 5, 8, and 12 stories. The researchers determined that the failure of reinforced concrete structures during the earthquake in Boumerdes city can be attributed to the quality of the materials employed, as well as the prevalence of weak column and strong beam construction in Algeria. Vivinkumar and Karthiga [9] conducted a comparative analysis of Force Based Design (FBD) and Direct Displacement Based Design (DDBD).

Based on FBD, DDBD, FEMA 356 [4], and IS 1893 [10], they looked at and created 2D skeletal frames with four, eight, and twelve stories. The authors came to the conclusion that the proportionally DDBD structure works well across all structural parameters and that the produced design behaved better and was safer than FBD buildings.

Mouzzoun et al. [11] evaluated the seismic behavior of a five-story reinforced concrete building in line with the seismic regulations of Morocco. It was discovered that the structure is susceptible to major earthquakes, yet it functions satisfactorily under moderate levels of danger.

Chaudhari and Dhoot [12] conducted an analysis and design of a four-story reinforced concrete (RC) structure in accordance with the Indian Standard code IS 456 [13]. The assessment of the building's performance level in terms of life safety was also carried out. The analysis was conducted in compliance with ATC 40 [5] and FEMA 273 [14]. It was discovered that the building's performance level meets the expected assumption. Li et al. [15] assessed the suitability and precision of POA in comparison to time history analysis (THA) for RC ductile frames subjected to multiple loads shaking table tests.

It was discovered that the POA consistently underestimated the response of the structure when the structure experienced severe damage and was close to collapsing. Kunnath [16] examined the factors to be taken into account when doing POA analysis under seismic activities, specifically focusing on nonlinear modeling issues.

2. Pushover Analysis Methodology

Theoretically, nonlinear dynamic analysis [17–22] is the best method for addressing this issue; however, it is extremely difficult to implement in practice due to the need for time history of ground motion data and detailed hysteretic behavior of structural member, both of which are highly unpredictable. Instead, this analysis is best reserved for research and for critically important structure design.

Assuming that the structure's response can be connected to that of an equivalent SDOF system is important to the pushover analysis. A time history response with a consistent mode shape indicates that the reaction is regulated by a single mode.

Neither assumption is right, but studies [14, 15] have shown that when the response of a structure with multiple degrees of freedom (MDOF) is dominated by a single mode, the assumptions can be used to provide accurate forecasts of the maximum seismic response.

Pushover analysis is a static nonlinear research performed to determine the building's pushover or capacity curve. Nonlinear static analysis must be performed to track the structures' gradual yielding. The structure is being loaded laterally. The weight is amplified until the desired building movement is achieved. This desired movement is the highest that can occur when the structure is shaken by a ground excitation of the specified level of intensity.

2.1. Target Displacement

The determination of building performance standards is contingent upon target displacement. In the past few years, numerous methodologies have emerged for the purpose of assessing target displacement. The approaches that can be identified are as follows:

1. The capacity spectrum method (ATC-40) [4].
2. The displacement method (FEMA-356) [5].
3. The displacement modification method (FEMA-440) [6].

2.1.1. Capacity Spectrum Method

To use the capacity spectrum approach, you'll have to transform your data from a capacity curve and demand response into a capacity spectrum (Sa vs Sd), which is an acceleration displacement response spectrum (ADRS) representation of your capacity curve [3]. Using the following equations, we can determine the modal participation factor (MPF1) and the modal mass coefficient (1), which are necessary steps in the process of transforming the capacity curve into the capacity spectrum.

$$\mathrm{MPF}_1 = \frac{\sum m_i\ \phi_{i1}}{\sum m_i\ \phi_{i1}^2} \quad (1)$$

$$\alpha = \frac{[\sum m_i\ \phi_i]^2}{[\sum_{i=1}^{N} mi][\sum_{i=1}^{N} m_i\ \phi_{i1}^2]} \quad (2)$$

Where mi is the floor-specific mass, φi1 is the amplitude of mode 1 on floor i, and N is the total floor number. The capacity curve is then solved for at each location to obtain Sa and Sd:

$$\frac{S_a}{g} = \frac{V_b}{w}\frac{1}{\alpha} \quad (3)$$

$$S_d = \frac{\Delta_{\mathrm{roof}}}{\mathrm{MPF}_1 \Phi_{\mathrm{roof1}}} \quad (4)$$

Specifically, Vb base shear, w total building weight, and Δroof roof displacement Using the following equation, we can get the value of Sd at each point along the curve, which is necessary for transforming a demand spectrum from Sa and T format to ADRS format.

$$S_d = \frac{T^2\ S_a}{4\pi^2} \quad (5)$$

Figure 2 shows the capacity spectrum approach. In order to estimate the effective damping and determine the required reduction in spectral demand, it is necessary to utilize a bilinear representation of the capacity spectrum. The determination of the trial performance point necessitates the establishment of a clear definition for the point (api , dpi).The performance point can be determined by identifying the intersection of the decreased response spectrum and the capacity spectrum at the estimated point.

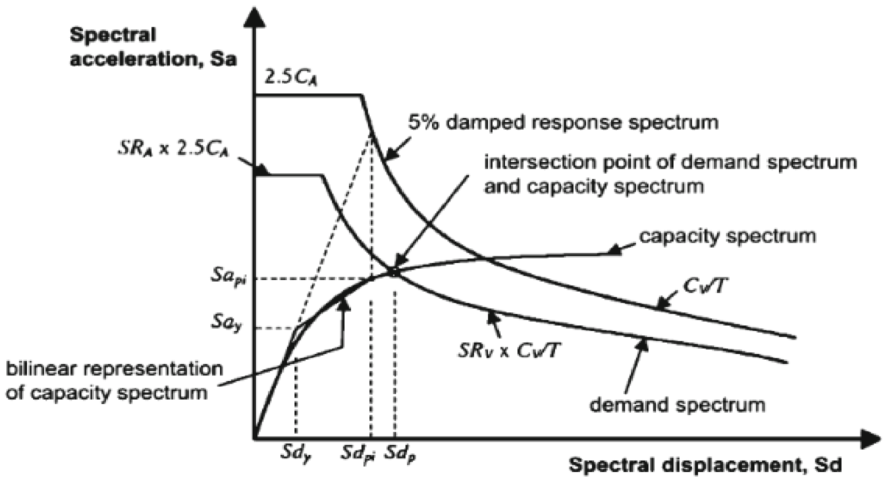


Figure 2. Performance point according to capacity spectrum method

2.1.2. Displacement Coefficient Method (FEMA 273/356)

FEMA 356 presents static nonlinear procedure. This approach is accomplished by adjusting the elastic response from SDOF equivalent with coefficient factors C_0, C_1, C_2, and C_3 [4] as follows:

$$\delta_T = C_0\, C_1\, C_2\, C_3\, S_a \left(\frac{T_e}{2\pi}\right)^2 \, g \qquad (6)$$

where **T_e** is the effective fundamental period; **C_0** is the modification factor that converts the spectral displacement of an equivalent SDOF system into the roof displacement of the building MDOF system; **C_1** is the modification factor that converts the expected maximum inelastic displacement into the displacement calculated for linear elastic response; **C_2** is the modification factor that represents the effect of pinched hysteretic shape, stiffness degradation, and strength deterioration on the maximum displacement response; and **C_3** is the modification factor that converts represent increased displacement due to dynamic **$P - \Delta$** effect. **S_a** response spectrum acceleration. **g** acceleration of gravity.

Figure 3 displays the target displacement (**δ_T**) calculation using the displacement coefficient approach. Beginning with an inelastic situation, an effective fundamental period is determined. The maximum spectral acceleration (**S_d**) is represented by the effective fundamental period, which is the linear stiffness of the comparable SDOF system. After that, you can use Eq. 6 to define your desired angular displacement.

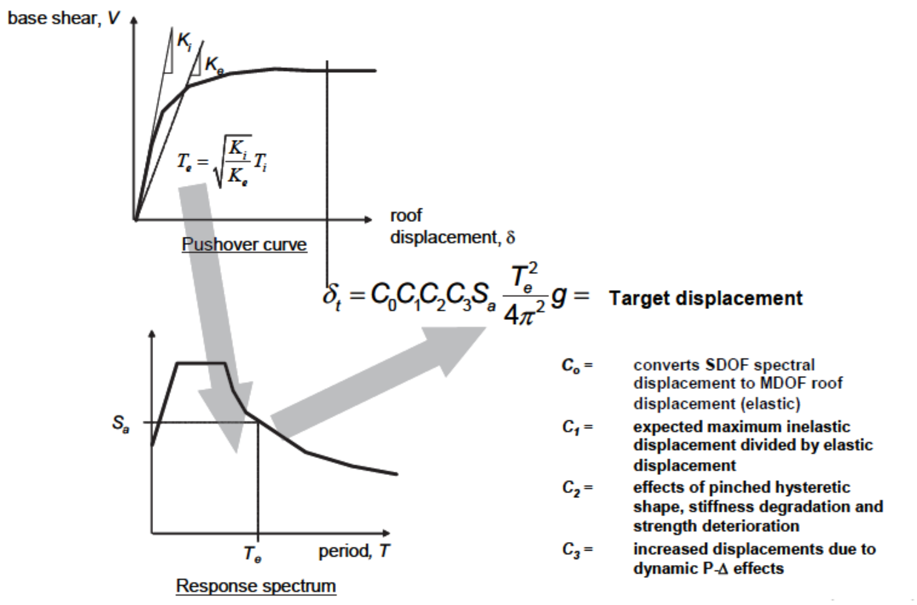


Figure 3. Performance point according to the displacement coefficient method

2.1.3. Displacement Modification (FEMA 440)

The previous two approaches may yield diverse outcomes. The FEMA 440 via ATC-55 Project resulted in the creation of FEMA 440, a publication focused on enhancing procedures for nonlinear static seismic analysis [6]. This methodology incorporates an equation that has resemblance to the displacement coefficient approach in order to ascertain the target displacement (δ_T). Nevertheless, there exist some alterations that can be employed to ascertain the values of C_1 and C_2, which are outlined as follows:

$$C_1 = 1 + \frac{R-1}{aT_e^2} \qquad (7)$$

$$C_2 = 1 + \frac{1}{800}\left(\frac{R-1}{T_e}\right)^2 \tag{8}$$

2.2. Nonlinear Plastic Hinge

The analysis of pushover requires the establishment of the force deformation curve for the critical section of beams and columns, as outlined in reference [6]. The curve depicted in Figure 4 is presented.

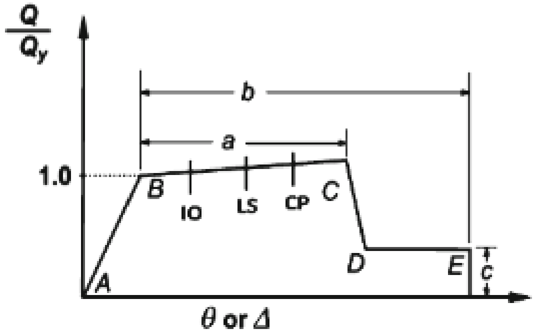


Figure 4. Force-deformation curve

This load deformation relation is characterized by a linear transition from the unloaded state (A) to the effective yield state (B). Afterward, there is a gradual loosening of stiffness from point B to point C. As the lateral load increases from point C to D, the resistance decreases suddenly, and finally, at point E, the resistance completely disappears. Most of the time, the slope of line B-C is between 0 and 10% of the original slope. When a member first gives out, it breaks along the C–D line. The member's residual strength is shown as a line from D to E. If you want to know how RC hinges rotate, you need to know these FEMA-specified points.

The points between B and C represent acceptance criteria for the hinge, which is immediate occupancy (IO), life safety (LS), and collapse prevention (CP).

2.3. Performance Limits

In general, there are two types of performance restrictions: global structural limits and local element limits [2, 5, 25, 26]. Specifications for the maximum allowable gravity load, lateral load resistance, and lateral deformation are established at the global level. If the capacity of a certain member to support gravity load is reduced, the structure must be able to shift that load to other elements. No more than 20% of the maximum resistance of the structure should be lost due to the building system's inability to withstand lateral loads. Lateral deformations need to be compared to the allowed ranges of motion, as stated in Table 2. Maximum drift is defined as the inter-story drift at the displacement of the performance point.

The limitations of elements are frequently affected by both non-structural and component damage. The determination of the reaction limits for structural elements, such as beams and columns, relies on the assessment of their plastic hinge rotation capacities. Tables 3 and 4 illustrate the deformation limits as per the ATC-40 guidelines, specifically focusing on the plastic hinge rotations of beam and column members within a reinforced concrete moment resistant frame. Hence, it is imperative to ascertain that the occurrence of flexural demands and shear failure does not precede the attainment of these rotation limits by a member.

Table 2. Deformation limits for each performance levels (ATC-40)

Immediate occupancy	Damage control	Life safety	Structural stability
0.01	0.01–0.02	0.02	0.33 Vi/Pi

Table 3. Plastic rotation limits for RC beams controlled by flexure (ATC-40)

$\frac{\rho-\rho'}{\rho_{bal}}$	Trans. Reinf.	$\frac{V}{b_w d\sqrt{f'_c}}$	Modeling parameter			Plastic rotation limit		
			a	b	c	Immediate occupancy	Life safety	Structural stability
≤0.0	C	≤3	0.025	0.05	0.2	0.010	0.020	0.025
≤0.0	C	≥6	0.020	0.04	0.2	0.005	0.010	0.020
≥0.5	C	≤3	0.020	0.03	0.2	0.005	0.010	0.020
≥0.5	C	≥6	0.015	0.02	0.2	0.005	0.005	0.015

Table 4. Plastic rotation limits for RC columns controlled by flexure (ATC-40)

$\frac{P}{A_g f'_c}$	Trans. Reinf.	$\frac{V}{b_w d\sqrt{f'_c}}$	Modeling parameter			Plastic rotation limit		
			a	b	c	Immediate occupancy	Life safety	Structural stability
≤0.1	C	≤3	0.02	0.03	0.2	0.005	0.010	0.020
≤0.1	C	≥6	0.016	0.024	0.2	0.005	0.010	0.015
≥0.4	C	≤3	0.015	0.025	0.2	0.003	0.005	0.015
≥0.4	C	≥6	0.012	0.02	0.2	0.003	0.005	0.010

3. Nonlinear Structural Modeling

Using computer software to conduct the analysis procedure needs a good understanding of the basics of this procedure and choosing the most suitable method to get more accurate and trusted analysis results. Therefore, there are some fundamental steps should be done to carry out a pushover analysis in CSI-ETABS program following the performance-based engineering principles to identify the real behavior of the RC buildings. The most important aspects are summarized in the following subsections.

3.1. Materials nonlinearity

In pushover analysis which is a non-linear static procedure that may be affected by what is called "hysteretic behavior" of the material which describe the process of energy dissipation through deformation. Several different hysteresis models are available to describe the behavior of different types of materials. Each hysteresis model may be used for the following purposes:

- Material stress-strain behavior.
- Single degree-of-freedom frame hinges, such as M3 or P hinges. Interacting hinges, such as P-M3 or P-M2- M3.

For each material stress-strain relationship or component hinge, an action versus deformation curve defined the nonlinear behavior under monotonic loading (pushover load) in the positive and negative directions is presented by what is called "backbone curve" as described before.

3.2. Plastic hinges assignment to structural elements

Nonlinear behavior of the structure is assumed to occur within a structure when there are concentrated plastic hinges assigned to various structural members that contribute in lateral loads resistance.

The distribution of concentrated plastic hinges and its length "lp" for shear walls are introduced in ASCE 41-17 [26] as shown in Fig. 5. Furthermore, Fig. 6 explains the actual and idealized curvature distribution in a wall segment showing elastic and plastic rotation occurred.

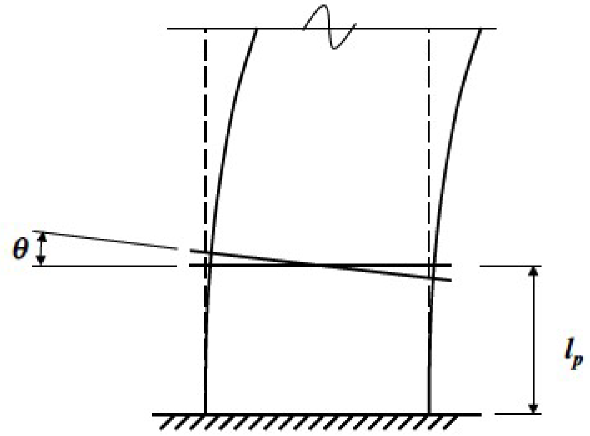


Figure 5. Plastic hinge rotation in shear wall

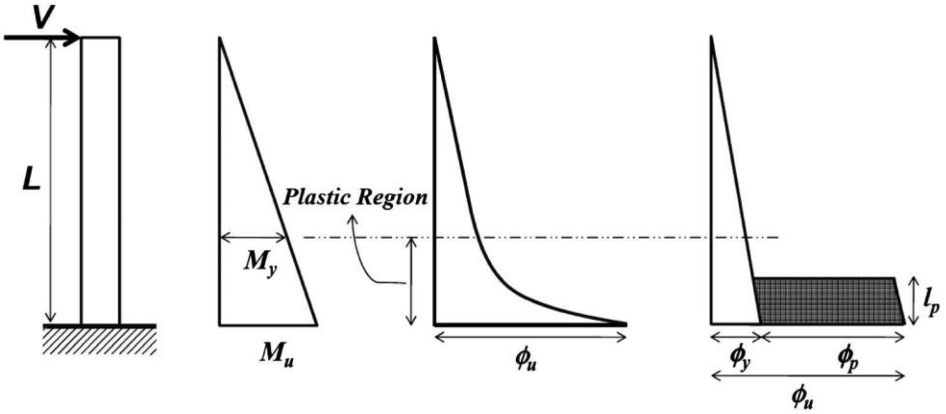


Figure 6. Actual and idealized curvature distribution in a shear wall.

For analytical models of shear walls and wall segments, the value of "lp" shall be set equal to 0.5 times the flexural depth of the element but less than one story height for shear walls and less than 50% of the element length for wall segments. The distribution of concentrated plastic hinges and its length "lp" for frame element are shown in Fig.7.

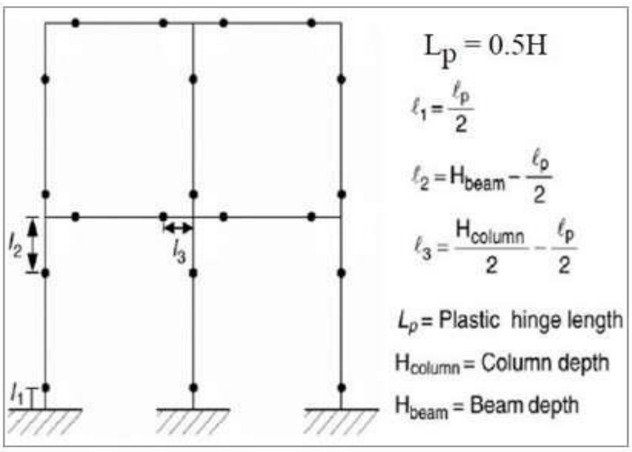


Figure 7. Plastic hinges distribution for beams and columns.

The default types include an uncoupled moment hinge “M3 hinges for beams”, an uncoupled axial hinge “P hinges for bracing members”, an uncoupled shear hinges and a coupled axial force and biaxial bending moment hinges “PM2- M3 hinges for columns” or “P-M3 hinges for walls”.

4. Verification Models

4.1. Model (1)

R.A. Hakim et al. (2014) the research works, and observations indicated that parts of the Kingdom of Saudi Arabia have low to moderate seismic regions. Major parts of buildings were designed only for gravity load and were poorly detailed to accommodate lateral loads. This study aims to investigate building performance on resisting expected seismic loadings. Two 3D frames were investigated using pushover analysis according to ATC-40. One was designed according to a design practice that considers only the gravity load and the other frame was designed according to the Saudi Building Code (SBC-301). Results showed that the building designed considering only the gravity load was found inadequate. On the other hand, the building designed according to SBC-301 satisfies the Immediate Occupancy (IO) acceptance criteria according to ATC-40.

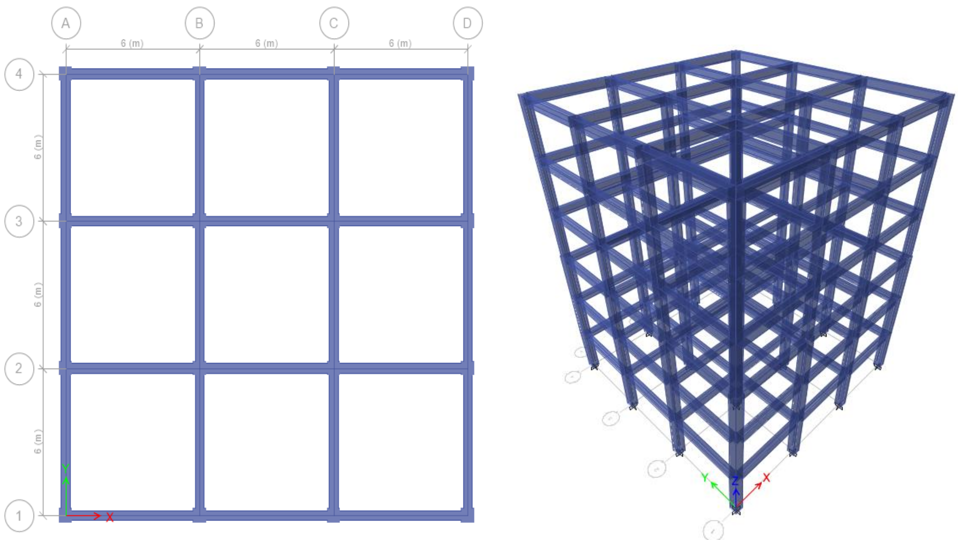


Figure 8. Structure layout and 3D model (Etabs)

A comparison between Etabs analytical and Sap analytical load-displacement curves for all reinforced concrete frames is presented in Figures (8). The figure shows that the analytical load-displacement curves are matching the analytical load-displacement curves. The quadratic curve also refers to the analytical results but using quadratic meshing instead of linear meshing, considering that using quadratic meshing in the analysis increases the accuracy of results. As shown in Figure (9), using quadratic meshing make the error smaller. Table (5) shows the comparison between Etabs analytical and Sap analytical values of ultimate flexural strength.

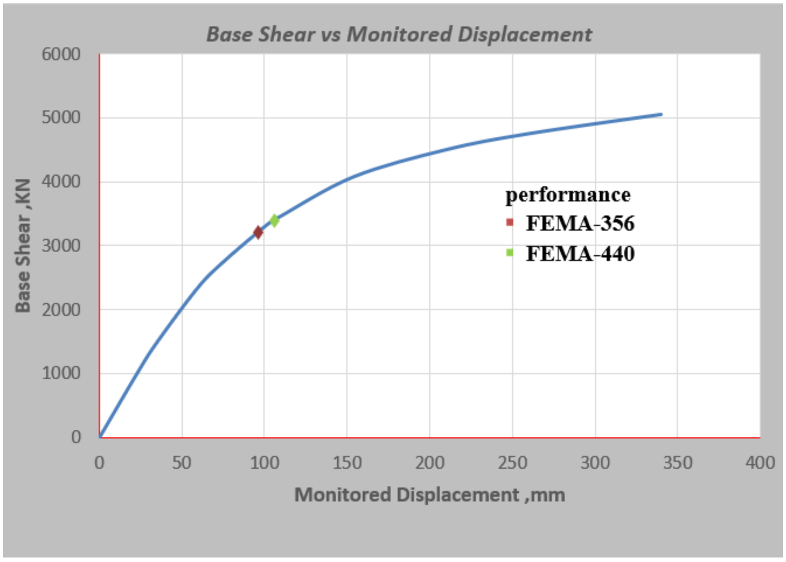


Figure 9. Modeling Results for reinforced concrete (RC) frames specimens

Table 5. Results for model -1 (Pushover analysis)

Model (2)	Old Analytical (shear)	New Analytical (shear)	% Error	Old Analytical (displacement)	New Analytical (displacement)	% Error
	Q_{ult} (KN)	Q_{ult} (KN)		Δ_{ult}(mm)	Δ_{ult}(mm)	
FEMA 356	3167.20	3250	2.60 %	101.21	97.79	3.38 %
FEMA 440	3198.31	3420.50	6.92 %	110.32	106.41	3.54 %

4.2. Model (2)

S. Z. Korkmaz et al. 2010 tested one bay, two-story bare RC specimen with no infill wall under a reversed cyclic loading. The goal of this test was to report on an experimental study about the Turkish EQ Code on proposed strengthening method. The specimens were subjected to lateral load that simulate the seismic action at the story level. Cycles were named as forward and backward cycles. Also, axial load was applied to top of columns. Dimensions and details of test specimens are presented in Figure (10). Figure (11) shows the ETABS analysis model conducted for this model and shows plastic hinges formation & crushing in concrete.

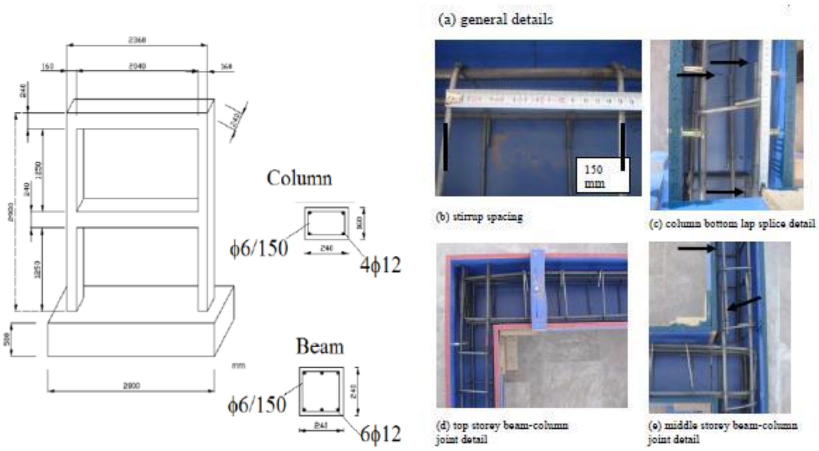


Figure 10. Dimensions and details of test specimens

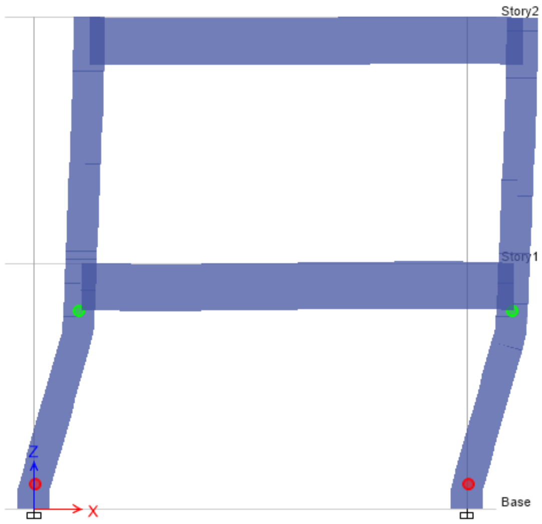


Figure 11. Plastic hinges formation & crushing in model

A comparison between experimental and analytical load-displacement curves for all reinforced concrete frames is presented in Figures (10). The figure shows that the analytical load-displacement curves are matching the experimental load-displacement curves. The quadratic curve also refers to the analytical results but using quadratic meshing instead of linear meshing, considering that using quadratic meshing in the analysis increases the accuracy of results. As shown in Figure (12), using quadratic meshing make the error smaller. Table (6) shows the comparison between experimental and analytical values of ultimate flexural strength.

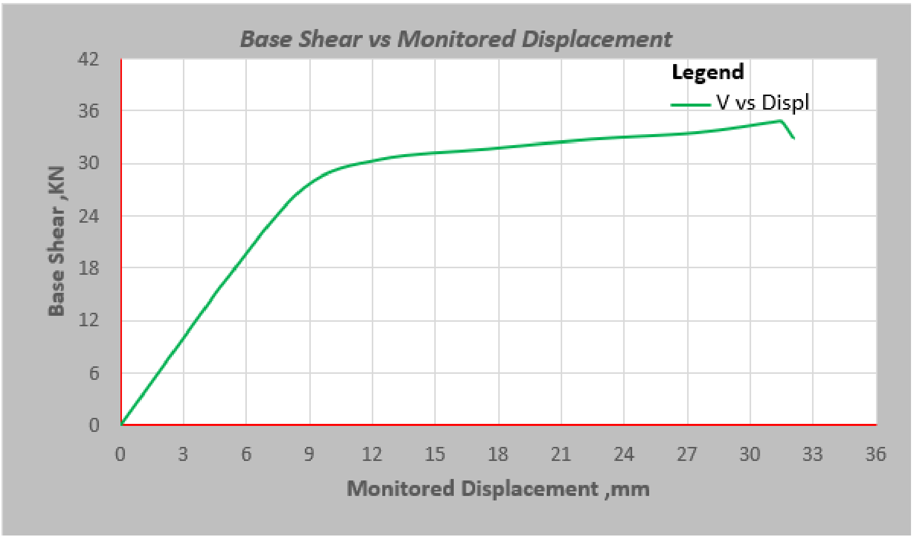


Figure 12. Modeling Results for reinforced concrete (RC) frames specimens

Table 6. Results for model - 2 (Pushover analysis)

Model (1)	Experimental (shear)	Analytical (shear)	% Error	Experimental (displacement)	Analytical (displacement)	% Error
	Q_{ult} (KN)	Q_{ult} (KN)		Δ_{ult}(mm)	Δ_{ult}(mm)	
1	34.51	34.77	0.75 %	34.75	31.50	16.56 %

5. Description of Buildings

This study investigates the seismic behavior of a multi-story reinforced concrete frame building consisting of 15, 20 and 25 floors with 2 basements for commercial use. The building has 7-spans for two-story basement and 5-spans for typical story of 8 m in both directions as shown in figure 13. The typical floor height is 3.6 m, except the basement floor height, which is considered to be of 3.3 m, which represents a typical building constructed in the new administrative capital, Egypt. The cross-sections of the columns are installed from the basement to the roof of the building, which is a common construction practice in Egypt.

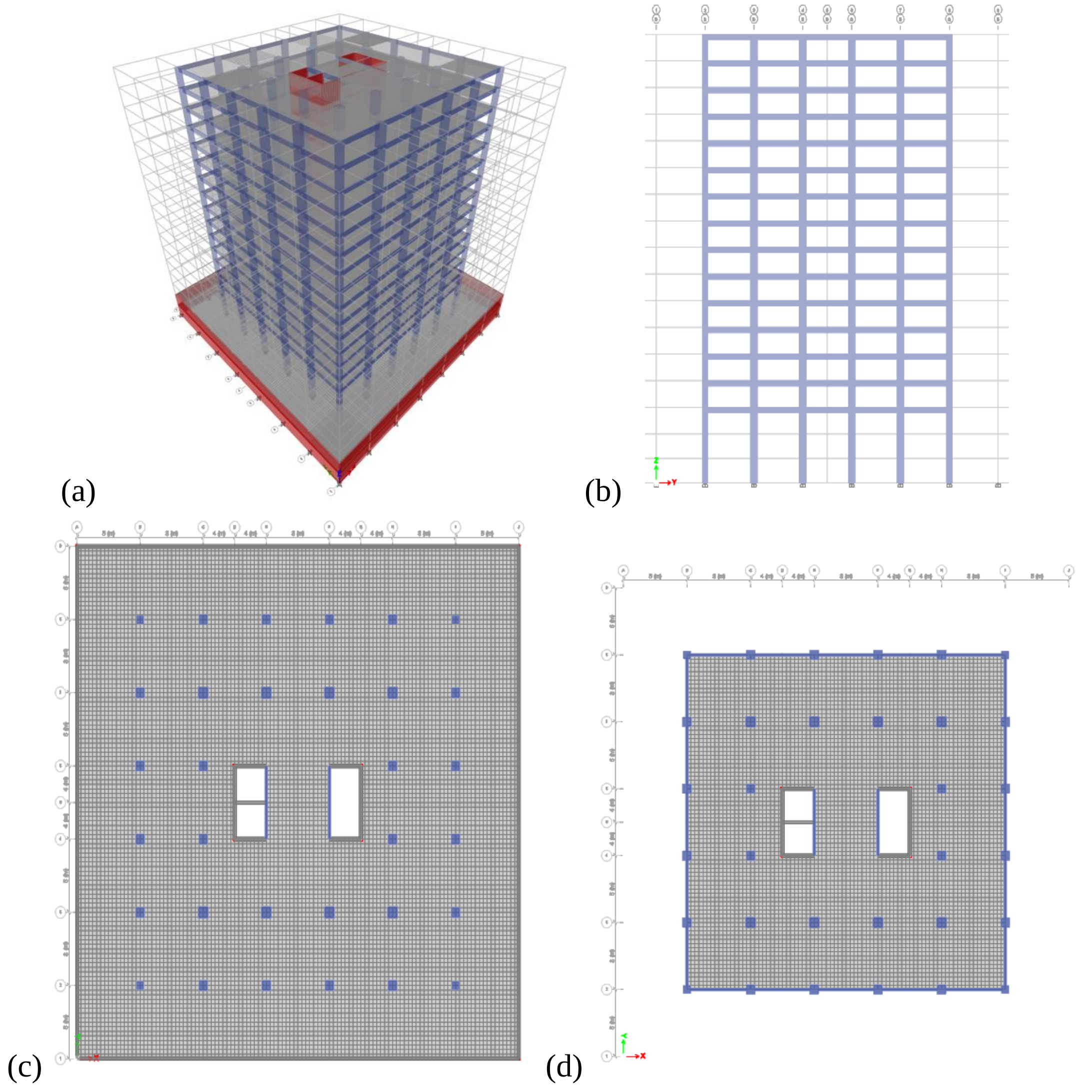


Figure 13. Schematic representation of (a) building 3D view (b) building elevation (c) floors plan of the 2 basement (d) and typical floors plan of the 15-storey framed building.

5.1. Material properties

High-grade steel with a yield strength of f_y = 360 N/mm2 is utilized for analysis and design, along with concrete with a characteristic strength f_{cu} of 25 N/mm2 after 28 days for beams, and the characteristic strength of concrete for columns and cores is 30 N/mm2. Using the formula Ec = 4400√fcu (ECP-201, 2020), we may calculate the modulus of elasticity given the specific weight of reinforced concrete, c = 25 kN/m3. Consider a value of 210 KN/mm2 for the elastic modulus of steel. We assume that the Poisson's ratios of concrete and steel are 0.2 and 0.3.

5.2. Section properties and reinforcement details

Cross sections for the interior, perimeter, and octagonal columns are assumed to be square. It is expected that the base of each column is permanently installed. Table 1 displays the dimensions of all frame members. The beams' chosen reinforcing ratios fall within the permitted range, with 1.25 percent and 0.3 percent being the upper and lower limits, respectively (ECP-201, 2020). Steel reinforcement ratios have been selected for the non-ductile columns to meet the requirements of the Egyptian code, which permits a range of between 4.0% and 0.8% for the proportion of steel reinforcement. The floor slabs in an RC building are 0.26 m thick, making them stiff floor diaphragms.

5.3. Gravity loads

The loads exerted on the RC building can be classified into two main categories: gravity loads and lateral loads. Gravity loads encompass both dead loads and live loads, while lateral loads encompass seismic loads. The designated values for the dead loads, specifically the weight of the flooring cover and the weight of the partitioning parts, are 2 kN/m2 and 2 kN/m2, respectively. The structural software package automatically calculates the self-weight of the structural parts as a component of the dead loads. As to the regulations outlined in the Egyptian code, the designated live load value for residential reinforced concrete (RC) buildings is 3 kN/m2, while the imposed load is set at 1.2 KN/h/m for all levels. The designated values for the dead loads, specifically the weight of the flooring cover, are 3.5 kN/m2. Additionally, the live load value for the roof floor of a residential reinforced concrete (RC) building has been determined to be 2k N/m2.

5.4. Lateral static loads equivalent to seismic loads

In the seismic design of buildings, the loads taken into consideration consist of the complete dead loads in addition to 50% of the live loads, as specified by the ECP-201 (2020). The seismic parameters required for calculating the base shear force of the building are provided in Table 7, which is based on the seismic characteristics specific to Cairo city. The pushover analysis method was utilized to conduct the structural assessment of the structure. Two distinct categories of loads were taken into account, specifically the GRAVITY load, which involves the application of predetermined vertical loads such as dead and live loads. The lateral loads applied in the x-direction and y-direction are denoted as PUSHX and PUSHY, respectively.

Table 7. The Seismic Design Data

S.N.	Parameters for design	Values
1.	Response Curve	1
2.	Importance factor γ_c	1
3.	Building location (zone)	Zone (3)
4.	Response reduction factor (R)	5
5.	Type of soil	Soil C
6.	Damping ratio	5%
7.	C_t factor	0.075
8.	Type of frame	Special moment resisting frame
9.	Number of floors	15-20-25
10.	Number of basements	2
11.	Basement floor height	3.30
12.	Typical floor height	3.60

6. Analytical Modeling

The ETABS software program has been utilized for the purpose of conducting the analysis. The nonlinear analysis was conducted using a three-dimensional model for each building [25]. The beams and columns are represented as nonlinear frame elements, incorporating lump plasticity at both the beginning and end of each element. The software ETABS version 19.1.1 has predefined hinge types for structural elements, specifically PMM hinges for columns and M3 hinges for beams, which are in accordance with the specifications outlined in FEMA-356.

The displacement controlled pushover analysis of the models is conducted after assigning all properties. The application of incremental loads follows the application of gravitational loads. The models are thereafter subjected to triangle and uniform load patterns, respectively, until the desired displacements are achieved. In order to achieve this objective, it is necessary to establish a series of steps that specify the displacement and target displacement at the roof.

In this study, the seismic responses of structures are evaluated using the design level earthquake in Cairo as specified in ECP-201 code. Demand response spectrum curve is constructed as shown in Fig. 14.

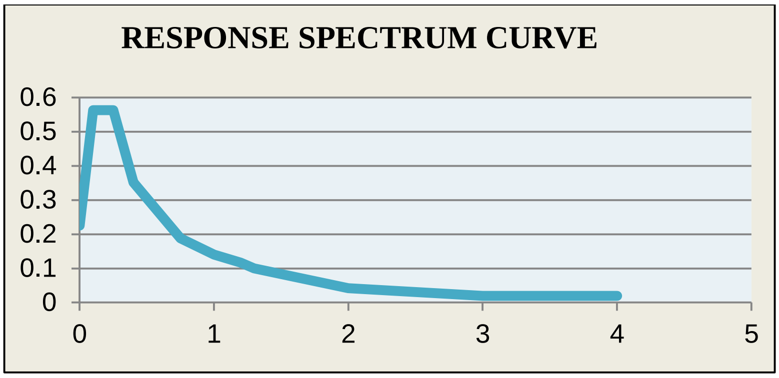


Figure 14. The demand spectrum

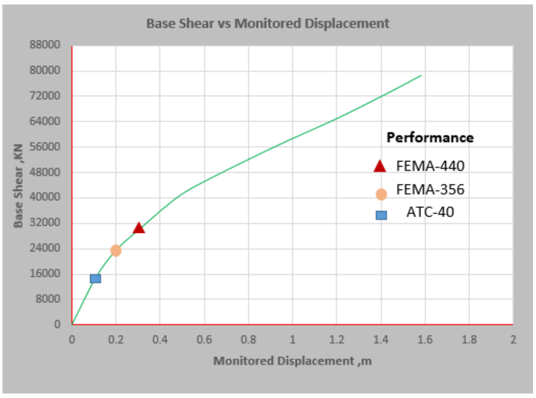
7. Result and Discussion

Figure 7 illustrates the pushover curves in the X direction for all buildings, whereas Figure 8 depicts the pushover curves in the Y direction for all buildings. The depicted curves illustrate the overall characteristics of the frame in terms of its stiffness and ductility. The rate of decline in the slope of pushover curves is shown to progressively decrease as the lateral displacement of the building increases. The progressive development of plastic hinges in the beam and column components of the structure is responsible for this phenomenon.

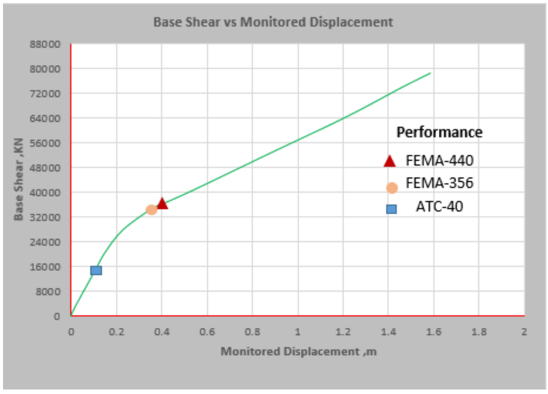
As previously stated in Section 2.3, pushover analysis takes into account two performance restrictions. The initial item pertains to the limit of structural stability. The limit state is established both at the overall structure level, specifically in relation to lateral load resistance, and at the individual story level, specifically in relation to the maximum inter-story drift ratio. The second limitation is determined by the plastic hinge rotation capacities acquired for each member, which are contingent upon their respective cross-sectional geometries.

The three methodologies, namely ATC-40, FEMA-356, and FEMA-440, employed for assessing performance points exhibit variations in their outcomes. The Capacity Spectrum Method, as outlined in the ATC-40 guidelines, identifies the performance point with the lowest level of structural capacity. Nevertheless, all three approaches consistently demonstrate that the margin of safety against collapse is substantial, and there exists ample strength and displacement reserves.

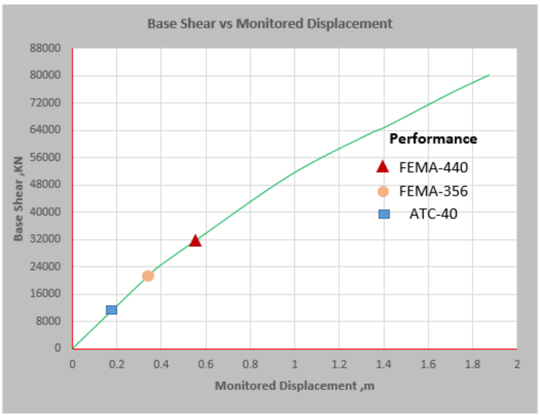
After each iteration of deformation, pushover analysis color-codes the plastic rotation hinges in the elements to show which ones have reached the IO, LS, and CP FEMA limit states. The creation of plastic hinges has been observed at a range of displacement values and performance trajectories, Fig. 15 Pushover curve in X direction at 0.15g and Fig. 16 Pushover curve in Y direction at 0.15g.



15 floors

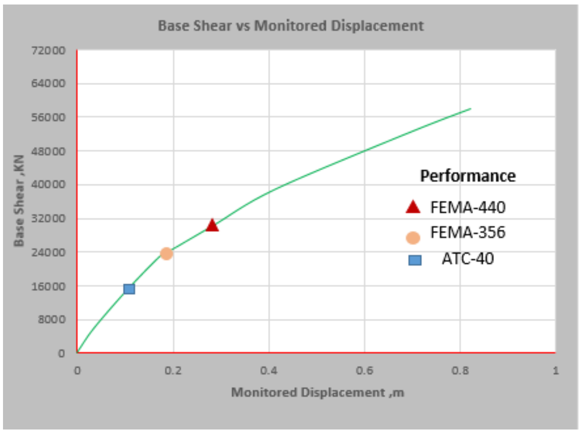


20 floors

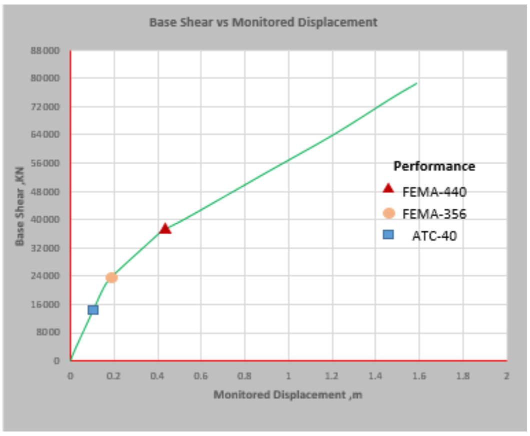


25 floors

Figure 15. Pushover curve in X direction at 0.15g (triangular load pattern)



15 floors



20 floors

25 floors

Figure 16. Pushover curve in Y direction at 0.15g (triangular load pattern)

8. Conclusion

Using pushover analysis, a study was conducted to determine how well buildings can withstand earthquakes. For this presentation, four buildings were studied, all of which adhere to the standards established by ECP-201. Pushover analysis has been conducted utilizing the capacity spectrum method (ATC-40), the displacement coefficient method (FEMA-356), and the displacement modification methods (FEMA-440).

The primary findings of the study provided above are as follows:

1. The use of pushover analysis offers a straightforward approach to observe the nonlinear response of a structure.
2. Distinct outcomes are obtained when employing the three methodologies (ATC-40, FEMA-356, and FEMA-440) for the determination of target displacement (δt). The Capacity Spectrum Method, as outlined in the ATC-40 guidelines, provides the minimum goal displacement, denoted as δt. Nevertheless, all three approaches consistently demonstrate that the margin of safety against collapse is substantial, and there exist ample reserves of strength and displacement.
3. The maximum story drifts range from 0.04 (0.01H) to 0.08 (0.02H), falling within the damage control (DC) category.
4. Since the worst-yielding elements have IO to LS levels, the structural damage to all buildings is still manageable.
5. In general, structures built in 2020 in accordance with ECP-201 are compliant with all three methods (ATC-40, FEMA-356, and FEMA-440).

The conclusions of the current study are constrained due to the consideration of only a single symmetry scheme inside a certain seismic zone. Furthermore, the pushover analysis is an approximation method that may not effectively capture dynamic phenomena with a high level of precision. The study has taken into account several factors, which have traditionally been treated as deterministic. However, it is important to acknowledge that these parameters exhibit considerable statistical variations. Therefore, a reliability-based framework is necessary to properly address these variations in the study

9. References

[1] *The Egyptian Code for Calculation of Loads and Forces in Structural and Building Work 2020. ECP-201, Housing and Building National Research Centre. Ministry of Housing, Utilities and Urban Planning, Cairo, Egypt.*

[2] *Mondal, A.; Ghosh, S.; Reddy, G.: Performance-based evaluation of the response reduction factor for ductile RC frame. J. Struct. Eng. 56, 1808–1819 (2013)*

[3] *Whittaker, A.; Hart, G.; Rojahn, C.: Seismic response modification factors. J. Struct. Eng. 125(4), 438–444 (1999).*

[4] *FEMA 356, "Prestandard and Commentary for the Seismic Rehabilitation of Buildings," Federal Emergency Management Agency, Washington, D.C., 2000.*

[5] *ATC-40.: Seismic Evaluation and Retrofit of Reinforced Concrete Buildings: Applied Technology Council (1996).*

[6] *FEMA 440, "Improvement of Nonlinear Static Seismic Analysis Procedures," Federal Emergency Management Agency, Washington, D.C., 2005.*

[7] *Eurocode 8, "Design of Structures for Earthquake Resistance," European Committee for Standardization, Brussels, 2004.*

[8] *A. Kadid and A. Boumrkik, "Pushover Analysis of Reinforced Concrete Frame Structures," Asian Journal of civil Engineering (Building and Housing), vol. 9, no. 1, pp. 75-83, 2008.*

[9] *R. V. Vivinkumar and S. Karthiga, "A Comparative Study on Force Based Design and Direct Displacement Based Design of Reinforced Concrete Frames," International Journal of Research in Engineering and Technology (IJRET), vol. 4, no. 4, pp. 46-53, 2015.*

[10] *IS 1893 (Part 1), "Indian Standard Code of Practice for Criteria for Earthquake Resistant Design of Structures," Bureau of Indian Standards, New Delhi, 2002.*

[11] *M. Mouzzoun, O. Moustachi, A. Taleb and S. Jalal, "Seismic Performance Assessment of Reinforced Concrete Buildings using Pushover Analysis," Journal of Mechanical and Civil Engineering (IOSR-JMCE), vol. 5, no. 1, pp. 44-49, 2013.*

[12] *D. J. Chaudhari and G. O. Dhoot, "Performance Based Seismic Design of Reinforced Concrete Building," Open Journal of Civil Engineering, vol. 6, no. 2, pp. 188-194, 2016.*

[13] *IS 456, "Indian Standard Code of Practice for Plain and Reinforced Concrete," Bureau of Indian Standards, New Delhi, 2000.*

[14] *FEMA 273, "NEHRP Guidelines for the Seismic Rehabilitation of Buildings," Federal Emergency Management Agency, California, 1997.*

[15] *S. Li, Z. Zuo, C. Zhai and L. Xie, "Comparison of Static Pushover and Dynamic Analyses using RC Building Shaking Table Experiment," Engineering Structures, vol. 136, no. 1, pp. 430-440, 2017.*

[16] *S. K. Kunnath, "Modeling of Reinforced Concrete Structures for Nonlinear Seismic Simulation," Journal of Structural Integrity and Maintenance, vol. 3, no. 3, pp. 137-149, 2018.*

[17] *Giannopoulos, P.I.: Seismic assessment of RC building according to FEMA 356 and Eurocode 8. In: 16th Conference on Concrete, TEE, ETEK, 21-23/10/2009.*

[18] *Fajfar, P.: Structural analysis in earthquake engineering—a break- through of simplified non-linear method. In: 12th European Con- ference on Earthquake Engineering, Paper Ref: 843 (2002).*

[19] *Martino, R.; Spacone, E.; Kingsley, G.: Nonlinear pushover analysis of RC structures. Adv. Technol. Struct. Eng., 1–8 (2000). Doi: 10. 1061/40492(2000)38*

[20] *Vijayakumar, A.; Babu, D.L.V.: Pushover analysis of existing rein- forced concrete framed structures. Eur. J. Sci. Res. 71(2), 195– 202 (2012)*

[21] *Poluraju, P.; Rao, N.: Pushover analysis of reinforced concrete frame structure using SAP 2000. In: International Journal of Earth Science and Engineering ISSN 0974-5904, Volume 04, No 06 SPL, pp. 684-690 (2011)*

[22] *Elnashai, A.S.: Advanced inelastic static (pushover) analysis for earthquake applications. Struct. Eng. Mech. 12(1), 51–69 (2001)*

[23] *Favvata, M.J.; Naoum, M.C.; Karayannis, C.G.: Seismic evaluation of infilled RC structures with nonlinear static analysis procedures. In: Proceedings of the 15th World on "Earthquake Engineering" Lisbon, Portugal, 24–28 September (2012)*

[24] *Favvata, M.J.; Naoum, M.C.; Karayannis, C.G.: Limit states of RC structures with first floor irregularities. J. Struct. Eng. Mech. 47(6), 791–818 (2013)*

[25] *ETABS User's Manual. 2013. Integrated Building Design Software. Computer and Structure Inc. Berkeley, USA.*

[26] *ASCE/SEI 41-17, "Seismic Evaluation and Retrofit of Existing Buildings," American Society of Civil Engineers, Reston, Virginia, 2017.*