

# Performance Based Seismic Design of Two RC High-Rise Buildings

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**Abstract:** There is a need to create a new generation of rehabilitation and design processes that integrate performance-based engineering principles. These include evaluating available capacities and structural strength then comparing them to deformation demands related to acceptable performance levels. This paper concerns with structural non-linear static analysis procedure (NSP) to investigate the performance of reinforced concrete (RC) buildings under seismic hazard. Two case studies of constructed high-rise buildings were analyzed using response spectrum analysis (RSA) and pushover analysis (POA) to evaluate the post-yield behavior, relative damage of the structure, story shears, roof displacement, story drifts, story moments, time period, plastic hinges formation, structure performance level and response modification factor (R). Based on the results of performance points for the two case studies, the buildings can sustain seismic base shear ranging from 85% to 65% of their ultimate capacity from POA in X- and Y-Directions. Furthermore, the calculated response modification factor differs from that prescribed by building codes.

**Keywords:** Performance-based design, Post-yield behavior, Pushover analysis, Performance levels, Non-linear static analysis.

## 1. Introduction

Performance Based Design (PBD) is a reliable methodology to the design of a new building or the assessment of an existing one that significantly reveals better results in comparison to conventional code addressed design procedures. In PBD, the designer works closely with the consultant to determine structural performance objective for serviceability and strength. The structure is then designed or assessed to make sure the predetermined objectives are accomplished. PBD is also becoming more essential considering recent intends to promote performance structural based design, evaluating systems at different phases up to their collapse limit, so that issues in relation to damage of structure and repair at predetermined "performance levels" can be highlighted. There are several procedures to define the building seismic performance depending on the prescribed standards. FEMA 356 [1] suggests displacement coefficient method (DCM) and ATC40 [2] discusses the capacity spectrum method (CSM). FEMA 440 [3] presented improvements to both the DCM and CSD. The Eurocode 8 [4] adopted the N2 method which exhibits a modified version of the CSM. Numerous researches revealed the behavior of RC structures when subjected to earthquake event. Kadid and Boumrkik [5] conducted POA on three buildings having framing resisting system with 5, 8, and 12 floors. They concluded that, the reasons for the reinforced concrete's failure during earthquake in Bumerdes city may be related to the used materials quality as well as the reality that the majority of buildings in Algeria are of the weak column and strong beam type. Vivinkumar and Karthiga [6] presents a comparative study for Force Based Design (FBD) and the Direct Displacement Based Design (DDBD). They analyzed

and designed 2D skeletal frames having four, eight, and twelve stories according to FBD, DDBD, FEMA 356 [1] and IS 1893 [7]. The authors concluded that, proportionally DDBD structure performs well throughout all structural parameters and delivered design had superior behavior and safe in comparison to FBD buildings. Mouzzoun et al. [8] assessed the seismic response of five-story RC building in accordance with the Moroccan seismic code. They found that, the building is vulnerable under severe earthquake, but performs well under moderate hazard. Chaudhari and Dhoot [9] analyzed and designed a four-story RC building according to IS 456 [10] and the performance level of life safety is checked. The analysis was done in accordance with ATC 40 [2] and FEMA 273 [11]. They found that, the building performance level conforms to the prescribed assumption. Li et al. [12] evaluated the applicability and accuracy of POA compared to time history analysis (THA) for RC ductile frame under multiple loading shaking table tests. They found that, the POA tended to significantly underestimate the response of structure when the structure suffered severe damage and near to collapse phase. Kunnath [13] discussed the nonlinear modeling considerations in performing POA analysis under seismic actions. A multistory frame subjected to lateral loads was presented to focus on differences that may arise as a result of modeling approaches. Shinde and Rangari [14] analyzed and designed a five-story RC building by applying response spectrum analysis. Utilizing pushover analysis in accordance with FEMA 356 [1], the behavior of plastic hinges and the performance point of this building are assessed. Gil-oulibé et al. [15] implemented the Performance-Based Seismic Design (PBSD) procedure on RC irregular frame utilizing POA. The results indicated that PBSD enhances the structure

performance and reveal better seismic load carrying capacity. Erdem and Karal [16] investigated the seismic performance of three, five and eight story existing and strengthened RC buildings using RC jacket of the internal columns and adding steel bracings. They found that, story drifts have significantly decreased in the strengthened buildings according to Turkish and American codes. Chaudhary and Chaudhary [17] presented a comprehensive literature survey for the limitations of design building codes, development of displacement and performance-based design as well as the unified performance-based design.

In this paper, two case studies of constructed high-rise buildings were analyzed using response spectrum analysis and pushover analysis to evaluate the post-yield behavior, relative damage of the structure, story shears, roof displacement, story drifts, story moments, time period, plastic hinges formation, structure performance level and response modification factor. The obtained results of performance points indicated that, the buildings can sustain seismic base shear ranging from 85% to 65% of their ultimate capacity from POA in X- and Y-Directions. Furthermore, the calculated response modification factor differs from that prescribed by building codes.

## 2. PERFORMANCE-BASED METHODOLOGY TOOLS

Performance Based Design (PBD) is a methodology of new buildings seismic designing or upgrading of existing ones, which incorporates a specific purpose to meet prescribed objectives for future earthquake performance. For assessment and design purposes in performance-based methodology, estimation of two major quantities is required. These are the seismic demand and the structure capacity. Developing a methodology is explained in Fig. 1.

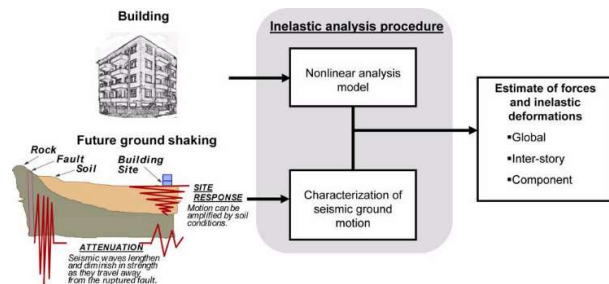


Fig 1: Developing a methodology of performance-based design [3].

### 2.1 Hazard Levels

A seismic hazard is typically characterized as the maximum possible earthquake that could possibly occur in the zone, together with of exceedance probability in each time period. For feasible design objectives, this is addressed in codes for practical design needs by response spectrum and the zone factor as stated in ATC 40 [2] and shown in Table 1.

TABLE 1: Performance objectives and hazard levels [2]

Event	Performance level	Hazard level	
	Limit state	Reoccurrence interval years	Probability of exceedance
Frequent	Operational	43	50% in 30 years
Occasional	Immediate Occupancy	72	50% in 50 years
Design earthquake	Life safety	475	10% in 50 years
Maximum earthquake	Collapse prevention	2475	2% in 50 years

### 2.2 Performance Levels

It is now recognized that some degree of damage is inherent during a major seismic event (hazard). The level of damage acceptable is normally identified through structure performance levels. There is an upcoming realization that the structure should be checked for sufficiency at respective performance stages. Performance levels according to FEMA 356 [1] and FEMA 273 [11] are collapse prevention (CP), life safety (LS), immediate occupancy (IO) and operational (O), in which life safety is the main purpose to reduce the structure's threats as clarified in Fig. 2.

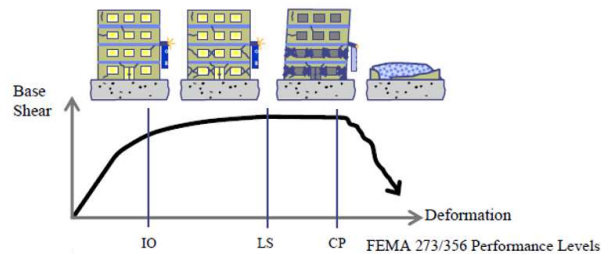


Fig 2: Illustration of performance levels and PBEE assessment process [1].

### 2.3 Force-Deformation Relationships and Performance-Based Evaluation

Force control and displacement control actions can be used to describe the applicable structure actions. For deformation-controlled behaviors, the force-deformation nonlinear relationship assigned to material stress-strain curves or elements plastic hinges must be defined for nonlinear static analysis indicating the post yield behavior and plastic deformation of structural members under monotonically increasing lateral loads. Some of such reliable and useful guidelines is FEMA 440 [3] and ASCE 41-17 [18] displaying these relationships according to Fig. 3 and Fig. 4, respectively.

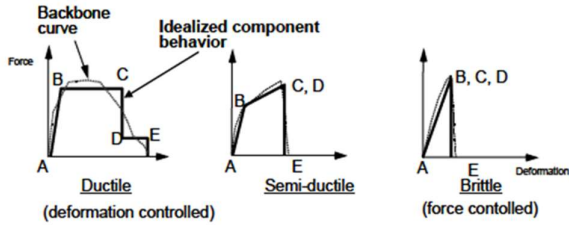


Fig 3: Idealized relationship for force-deformation component hinge [3].

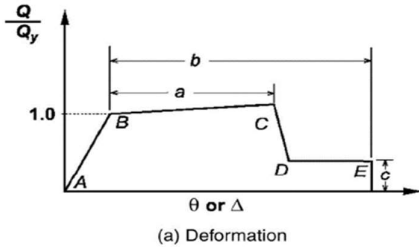


Fig 4: Force-deformation generalized relationship for components or concrete elements [18].

2.4 Performance-Based Evaluation Acceptance Criteria

Several performance levels are available for seismic PBD evaluation of structure components. (O, IO, LS, CP) are identified and assigned on the force-deformation nonlinear relationships appointed to material stress-strain curves or element plastic hinges, shown in Fig. 5. Generally, five points marked A, B, C, D, and E are used to describe the force-deformation action of a component hinge, The corresponding points to the prescribed performance levels are also stipulated as the “hinge acceptance criteria” that describe force-displacement or moment-rotation behavior of a component hinge.

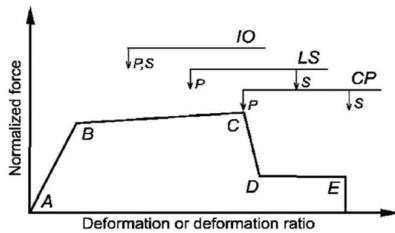


Fig5: Force-deformation relationship and acceptance criteria illustration [18].

3. PUSHOVER ANALYSIS PROCEDURE

Pushover analysis (POA) is a non-linear static procedure for seismic analysis of structures, the nonlinear load-deformation characteristics of individual components are directly incorporated throughout mathematical model that illustrates how a monotonically increasing lateral loads or directions accelerations indicating earthquake inertia forces are applied to the structure, consequently different structural members may successively yield under loads that increase incrementally, due to this the structure loses stiffness with each event until a target displacement is exceeded. POA development is shown in Fig. 6.

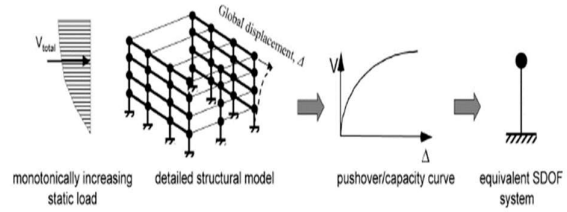


Fig 6: Schematic illustration for developing an equivalent SDOF system from a pushover procedure [3].

3.1 Pushover Analysis Methods

The common methods used for pushover analysis are Energy-Based POA methods, Conventional POA methods and Adaptive POA methods. In this paper, the conventional methods are used as a tool of performance-based assessment. Capacity Spectrum Method (CSM), Improved Capacity Spectrum Method, (ICSM), N2 method, Displacement Coefficient Method and Modal Pushover Analysis (MPA) can be classified as conventional POA methods.

3.1.1 Capacity spectrum method (CSM)

Freeman et al. [19] initially introduced this method. CSM is a technique for quick building seismic evaluation (Fig. 7). The methodology has subsequently been approved for use as a seismic design tool. The CSM, an equivalent linearization method, was described by ATC 40 [2]. The main assumption in equivalent linearization strategies is the estimation for peak total deformation for a SDOF nonlinear system from the linear elastic SDOF system peak deformation that has a time period and a greater damping ratio than that of the nonlinear system's initial values.

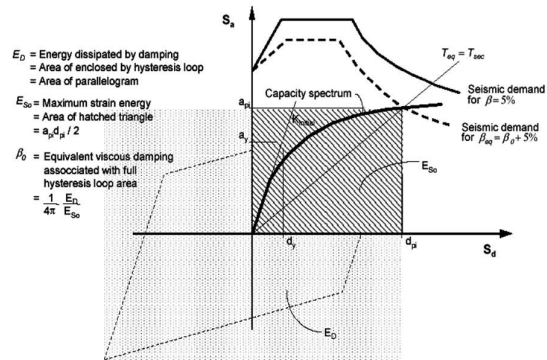


Fig 7: Estimation of the CSM approach.

3.1.2 N2 Method

Fajfar et al. [20] introduced the N2 approach as an alternative for the CSM method. N (Nonlinear analysis) & 2 (Two mathematical models are carried out). The N2 method is a variant of CSM based on inelastic demand spectra which established using reduction factors from a typical elastic spectrum.

3.1.3 Displacement coefficient method

This method is described in FEMA 356 [1] and FEMA 273 [11]. The inelastic behavior of structural materials

incorporated in a mathematical model which is displaced till reaching a target displacement. Also, ATC 40 [2] determined the internal deformations and forces for structural elements as shown in Fig. 8.

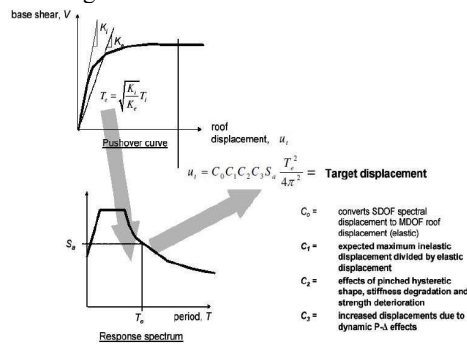


Fig 8: Procedure of DCM method [2].

### 3.2 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.

#### 3.2.1 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.

#### 3.2.2 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 “ $l_p$ ” for shear walls are introduced in ASCE 41-17 [18] as shown in Fig. 9.1. Furthermore, Fig. 9.2 explains the actual and idealized curvature distribution in a wall segment showing elastic and plastic rotation occurred.

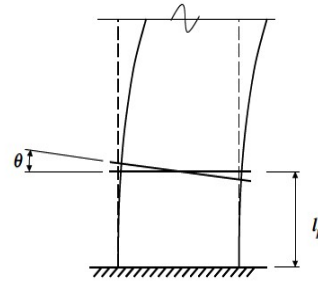


Fig 9.1: Plastic hinge rotation in shear wall

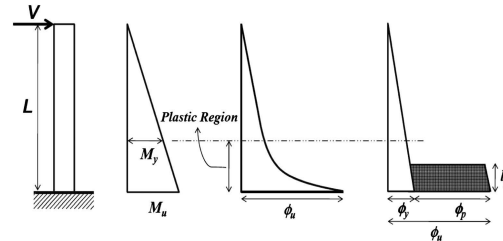


Fig 9.2: Actual and idealized curvature distribution in a shear wall.

For analytical models of shear walls and wall segments, the value of “ $l_p$ ” 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 “ $l_p$ ” for frame element are shown in Fig.10.

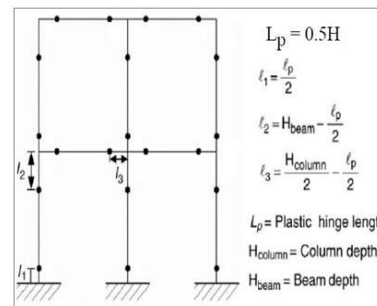


Fig 10: Plastic hinges distribution for beams and columns.

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 “P-M2-M3 hinges for columns” or “P-M3 hinges for walls”.

### 4. RESPONSE MODIFICATION FACTOR (R)

The R factor is a key seismic design tool, which stipulates the inelasticity level of the lateral load resisting system (LLRS) for an earthquake. Force reduction or response modification factor is used to minimize the structure's elastic response to the design response level. There are a different (R) values for various types of LLRS. In this paper, the R values are calculated for two case studies using the FEMA 356 [1]and ATC 63 [21] methods, then are compared with the R values mentioned in American and Egyptian building codes.

## 5. CASE STUDIES

Two high-rise buildings with different structural systems and heights which had already been built in United Arab Emirates and Egypt are investigated as case studies. For each building a 3D numerical model is generated using CSI-ETABS program. First, each case study is analyzed using the response spectrum method. Following that, performance-based procedure based on pushover approach is done to assess the capacity of these buildings under prescribed seismic hazard and results from the two procedures are compared to clarify the structure performance.

### 5.1 Case Study No. 1

#### 5.1.1 Building description

A 60-story RC building with an overall height of 260 m above the ground located in the UAE represents the first investigated case study. The resisting system for lateral loads is tube-in-tube structural system. Outrigger girders that connect the core to the outer columns were used at levels number 36 and 60. The steel reinforcement yield stress was 460 MPa. Concrete grades were C50 and C70 for horizontal and vertical elements, respectively. The structure loads were assigned according to UBC 97 [22]; the building was classified in zone 2A of seismic hazard with basic ground acceleration “0.15g”, soil category “SC” and response modification factor equals to 5.5. The developed 3D model and floor plans layouts are shown in Figs. 11 - 13.

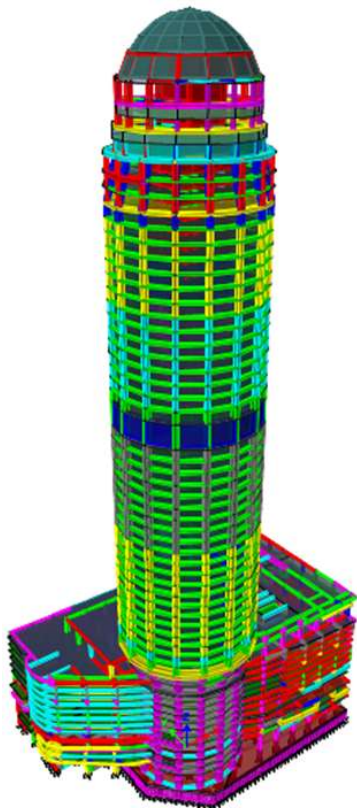


Fig 11: 3-D analysis model for case study No. 1

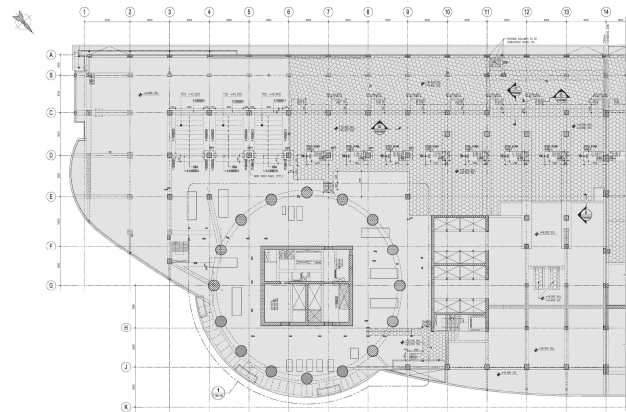


Fig 12: Podium floors layout

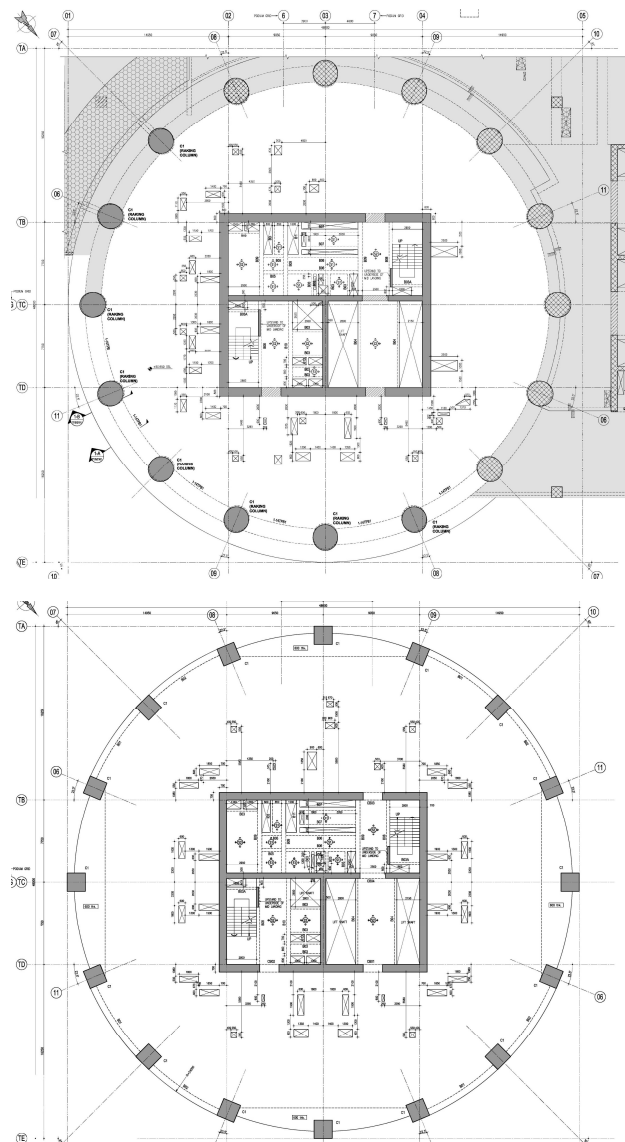


Fig 13: Typical floors layout

5.1.2 Output results

The global structure response to the prescribed seismic event represented in story shears, moments, displacements, drifts, time periods and plastic hinge formation, is shown in Figs. 14-22.

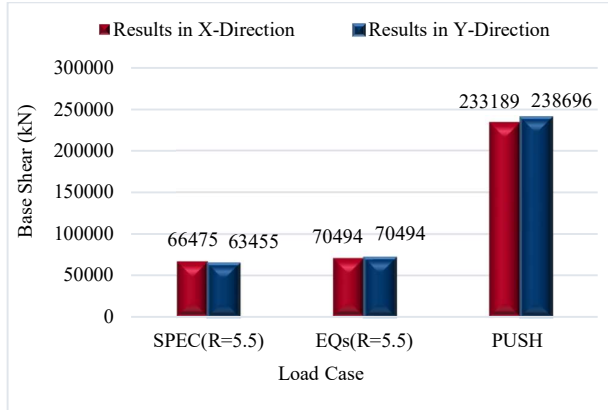


Fig 14: Comparison of base shear results

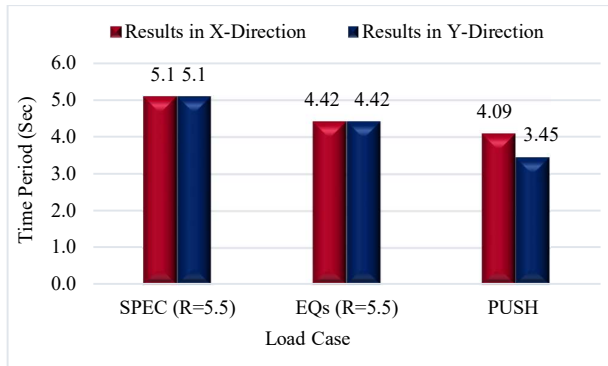


Fig 15: Comparison of time period results

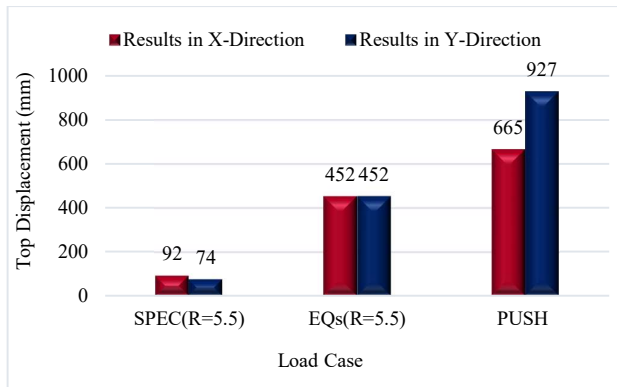


Fig 16: Comparison of top displacement results

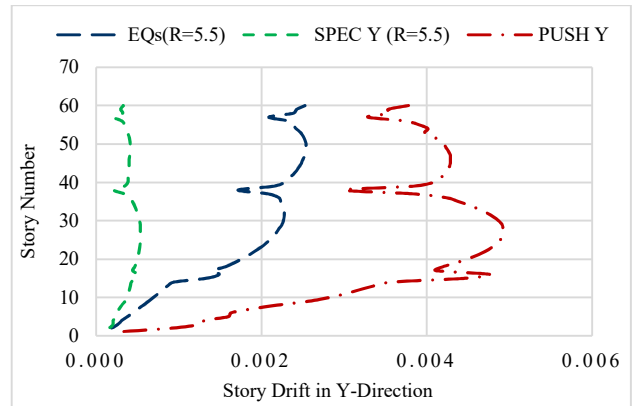
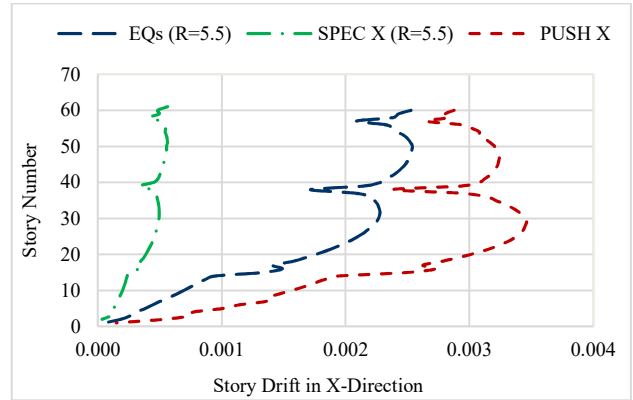


Fig 17: Story drifts

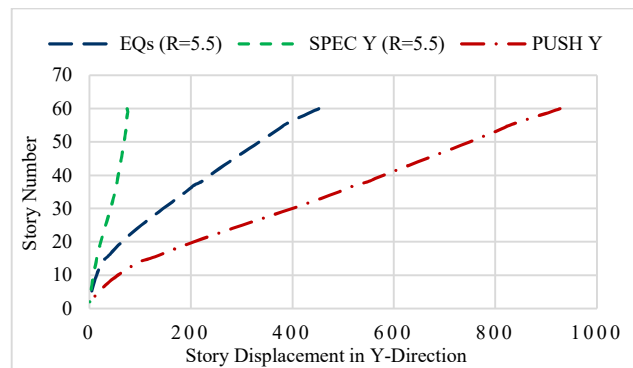
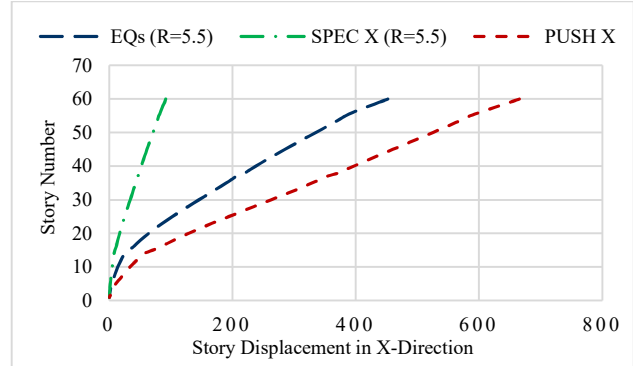


Fig 18: Story displacements (mm)

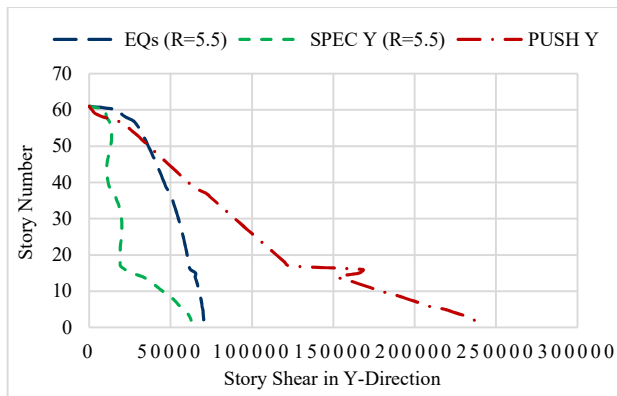
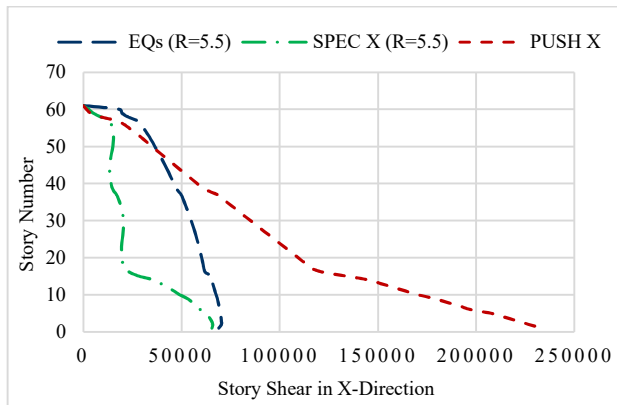


Fig 19: Story Shears (kN)

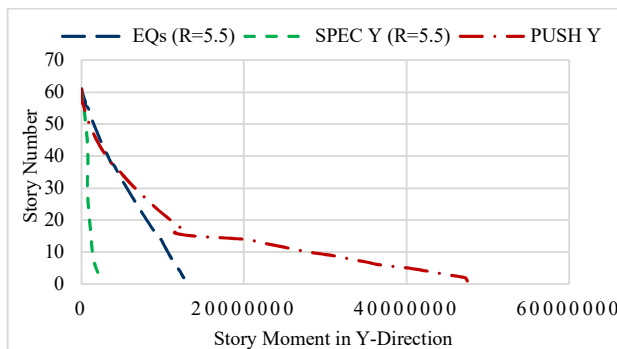
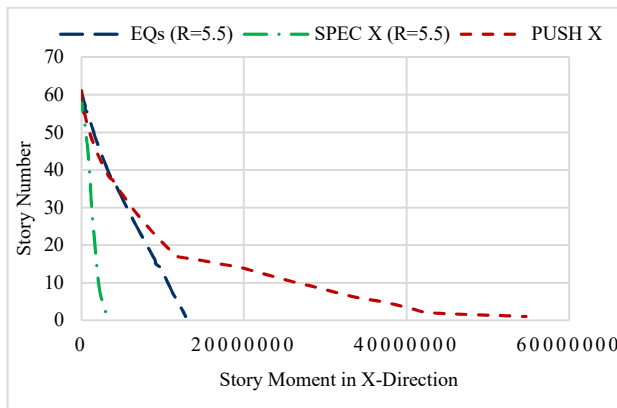


Fig 20: Story moments (kN.m)

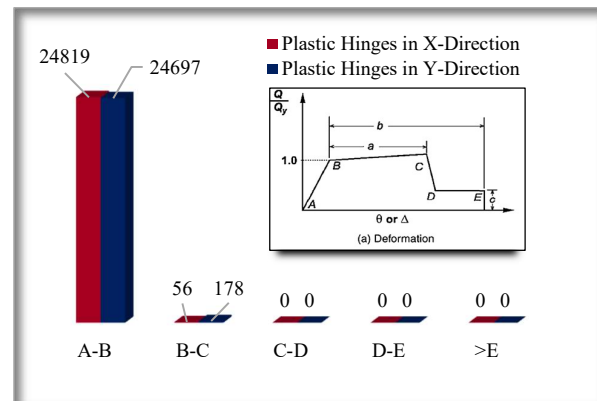


Fig 21: Plastic hinges related to deformation limits

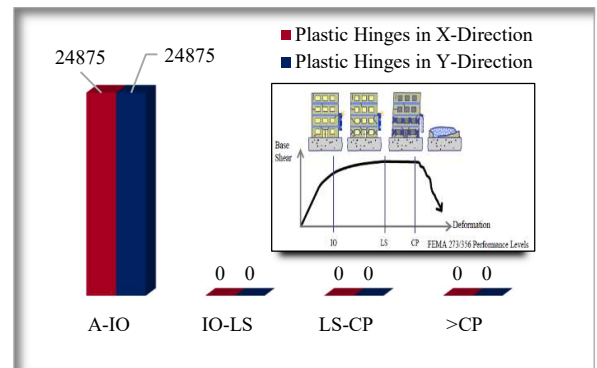


Fig 22: Plastic hinges related to performance levels

- Based on base shear results, the structure capacity in X-and Y-Directions from pushover analysis is about 3.40 times the design base shear from response analysis. Therefore, this building has a large safety margin till reach its ultimate capacity.
- The structure displacement in X-and Y-Directions from pushover analysis is about 1.5 to 2 times the displacement from response spectrum analysis leading to high displacement are reserved.
- According to the formation of plastic hinges in X-and Y-Directions, most of these hinges are formed in low deformation limits which ranging from A to B “pre-yield zone” in force-deformation relationship for plastic hinges.
- Also, plastic hinges are formed from A to IO “Immediate Occupancy” performance level clarifying that there is no significant damage would occur to structure and the structure can retain its original strength and stiffness.

5.1.3 Structure performance points

Structure performance points are described by base shear and corresponding displacement and obtained as a result of intersection of demand spectrum and capacity curve in spectral format between spectral acceleration “Sa” and spectral displacement “Sd”. The structure performance points in X- and Y directions according to FEMA 440 [3] are shown in Fig. 23 and Fig. 24.

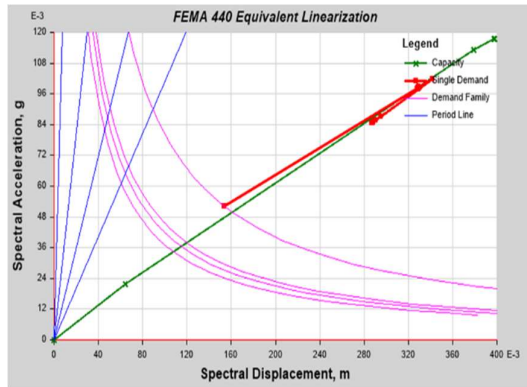


Fig 23: Performance point in X-direction (198771 kN and 567 mm)

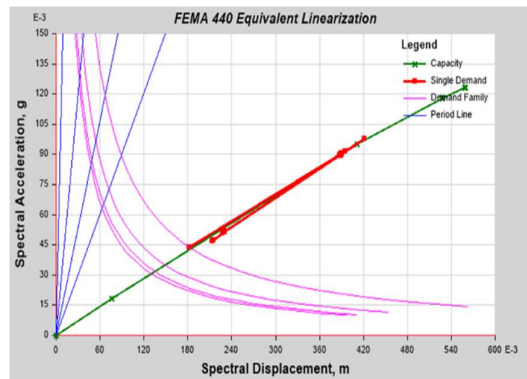


Fig 24: Performance point in Y-direction (177015 kN and 658 mm)

- According to performance point for structure in X-Direction, the structure can sustain seismic base shear equal to 198771 kN which represents about 85% of its ultimate capacity (233189 kN) from pushover analysis. Also, its displacement equals to 567 mm represents about 85% of its ultimate displacements (665 mm).
- According to performance point for structure in Y-Direction, the structure can sustain base shear equal to 177015 kN under prescribed seismic loads which represents about 74% of its ultimate capacity (238696 kN). And, its displacement equals to 658 mm represents about 71% of its ultimate displacements (927 mm).
- It can be concluded that, the obtained results for base shear and displacement are higher than those calculated using the design response spectrum

method. Consequently, obtained results according to design code revealing that there are enough strength and displacement reserved at structure performance point.

5.1.4 Response modification factor

The response modification factor (R) in both directions calculated using FEMA 356 [1] and ATC 63 [21] are given in Tables 2 and 3 respectively.

Table 2: R Values in X- and Y-Directions According to FEMA 356 Method

Direction	X-Direction	Y-Direction
Ki (kN/m)	349243	237475
Ke (kN/m)	349243	237475
Ti (sec)	3.45	4.09
$T_e = T_i \sqrt{\frac{K_i}{K_e}}$ (sec)	3.45	4.09
Sa (g)	0.0844	0.0712
W (kN)	3806621	3806621
Vy (kN)	39634	34905
Cm	1.0	1.0
$R = S_a / (V_y / W) * C_m$	8.11	7.76

Table 3: R Values in X and Y – Directions According to ATC 63 Method

Direction	X-Direction	Y-Direction
Vy (kN)	39634	34905
Ve (kN)	233189	238696
Vd (kN)	23780	20943
Δu (mm)	113	147
Δy (mm)	665	927
$\Omega = V_y / V_d$	1.667	1.667
$\mu = \Delta u / \Delta y$	5.885	6.306
Rμ	5.885	6.306
$R = \Omega \cdot R_\mu \cdot R_R \cdot R_\xi$	9.81	10.51

- According to calculated response modification factor “R” in both directions, the R values are greater than the value of 5.5 that allocated in design code UBC 97 [22]. This means that the structure has a higher ductility and higher ability for earthquake energy dissipation upon on its nonlinear behavior and its members ultimate capacity.



## 5.2 Case Study No. 2

### 5.2.1 Building description

A 16-story RC building with an overall height of 59 m above the ground built in Egypt represents the second investigated case study. The lateral load resisting system is a dual system. The concrete grades for vertical and horizontal elements were C30. The yield stress for steel reinforcement is 400 MPa. The building loads are assigned according to ECP 201 [23]. The building is classified in zone 3 of seismic hazard with basic ground acceleration “0.15g”, soil category “C” and response modification factor equals to 5.0. The developed 3D model and floor plans layouts are shown in Figs. 25–27.

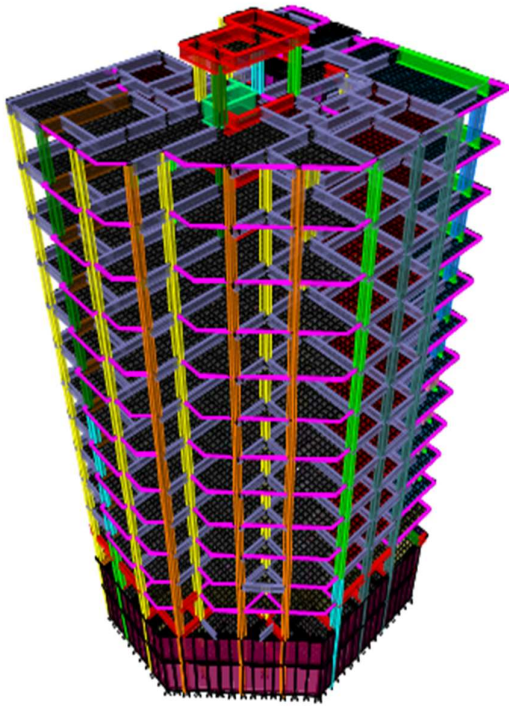


Fig 25: 3-D analysis model for case study No. 2

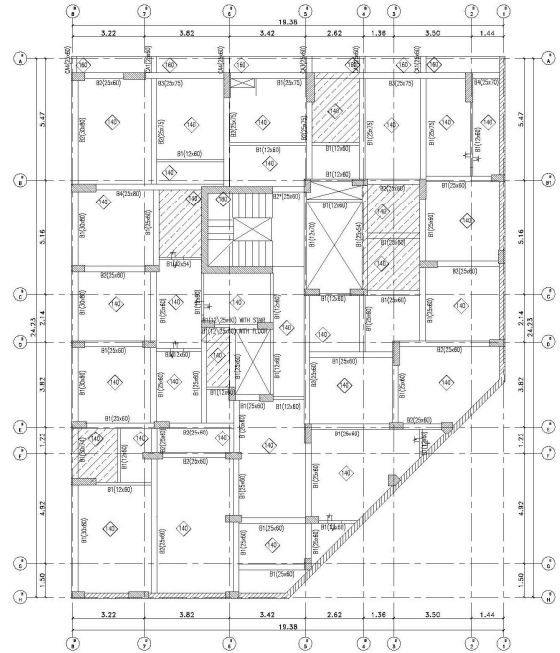


Fig 26: Basement floors layout

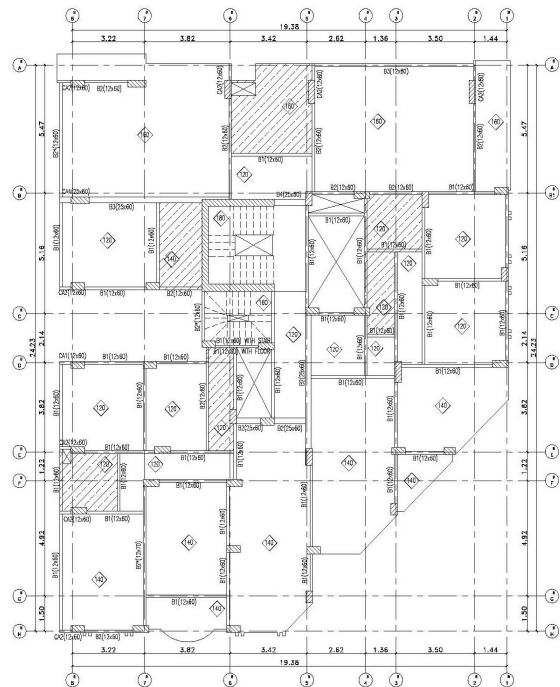


Fig 27: Typical floors layout

5.2.2 Output results

The structure global response to the prescribed seismic event represented in story shears, moments, displacements, drifts, time periods and plastic hinge formation, is shown in Figs. 28-36.

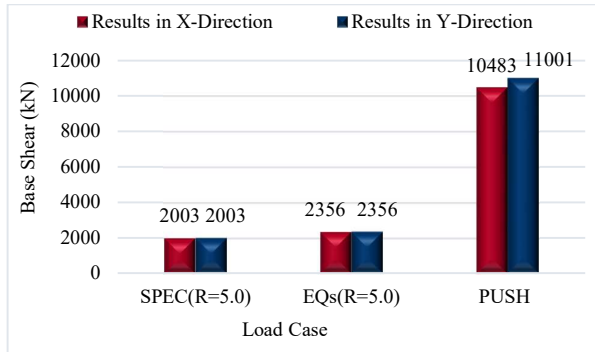


Fig 28: Comparison of base shear results

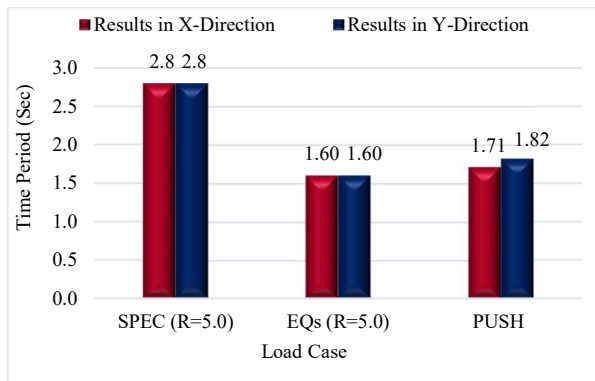


Fig 29: Comparison of time period results

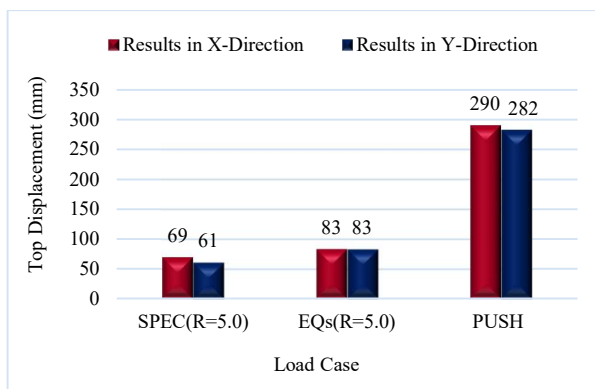


Fig 30: Comparison of top displacement results

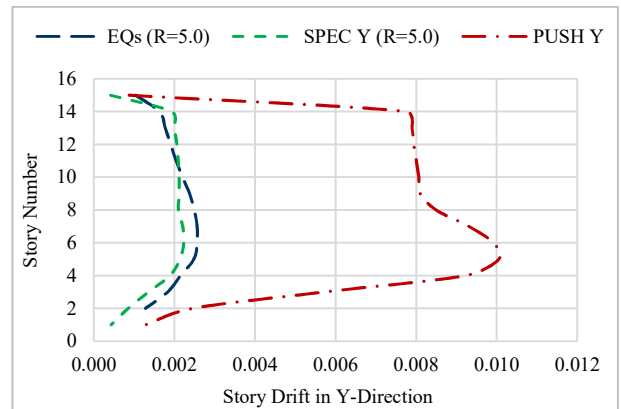
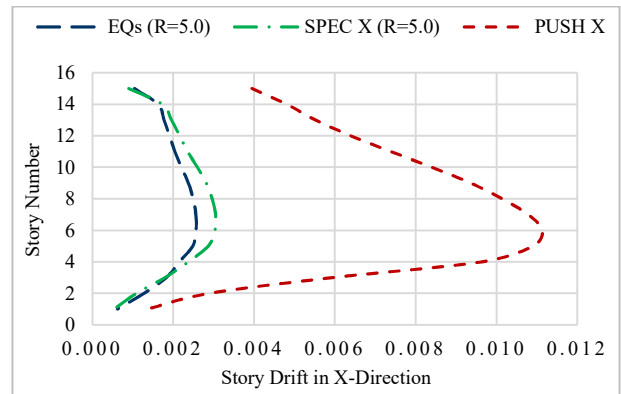


Fig 31: Story drifts

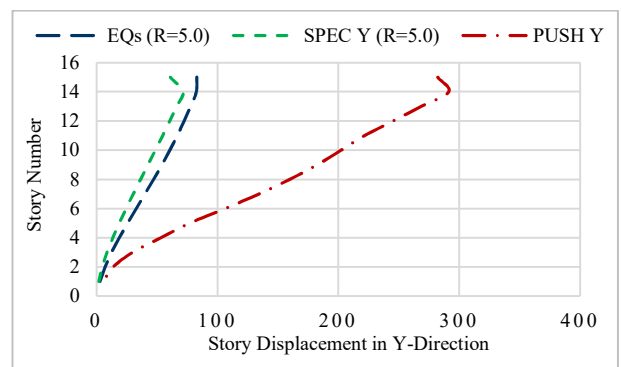
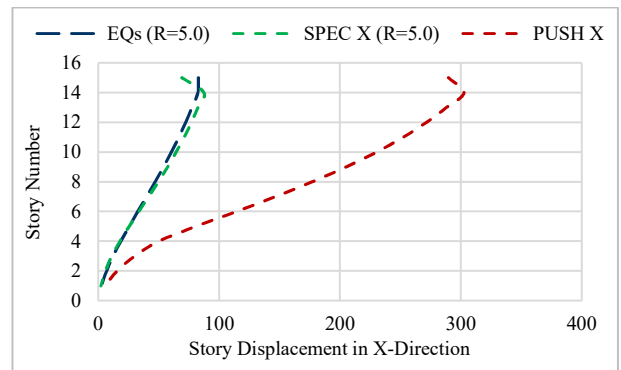


Fig 32: Story displacements (mm)

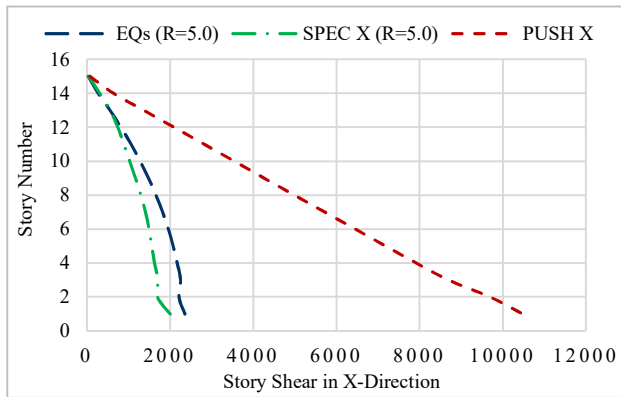


Fig 33: Story Shears (kN)

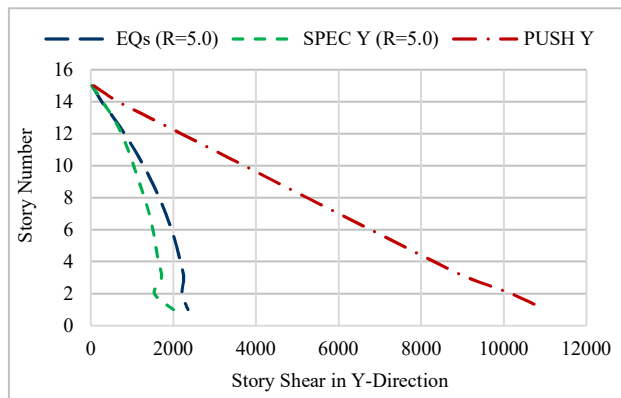


Fig 33: Story Shears (kN)

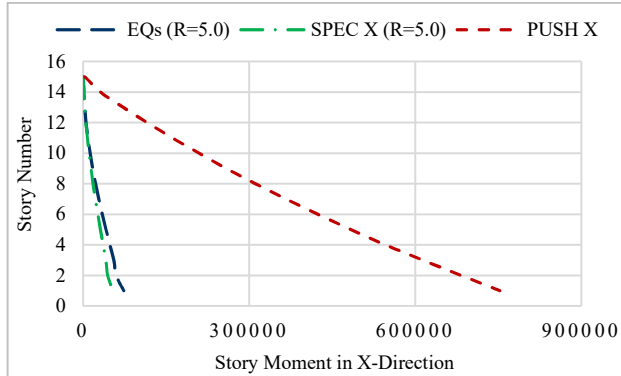


Fig 34: Story moments (kN.m)

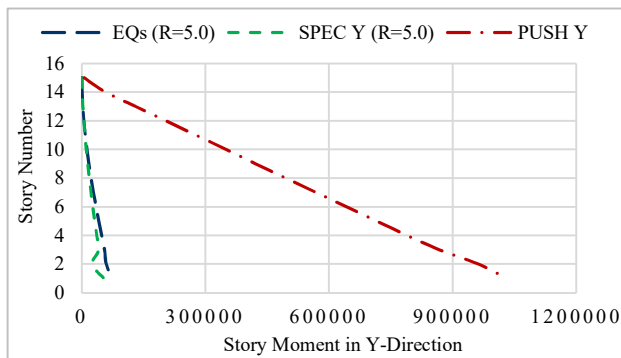


Fig 34: Story moments (kN.m)

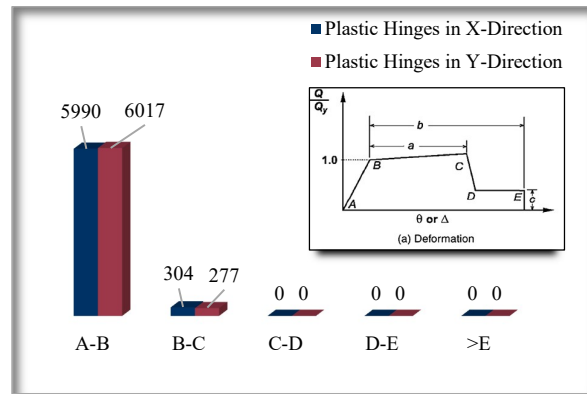


Fig 35: Plastic hinges related to deformation limits

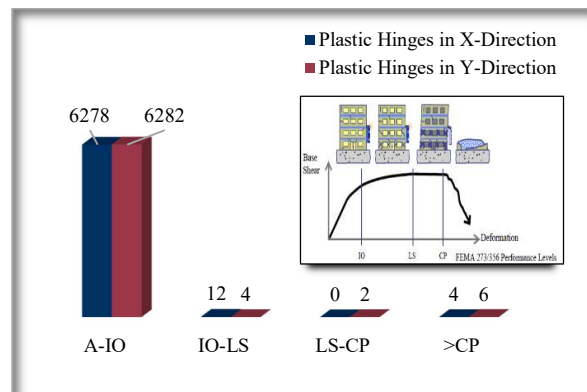


Fig 36: Plastic hinges related to performance levels

- Based on base shear results, the structure capacity in X-and Y-Directions from pushover analysis is about 4.50 to 4.80 times the design base shear from static analysis. Therefore, this building has a large safety margin till reach its ultimate capacity.
- The structure displacement in X-and Y-Directions from pushover analysis is about 3.50 to 3.40 times the displacement from static analysis leading to high displacement reserves.
- According to the formation of plastic hinges in X- and Y-Directions, most of these hinges are formed in low deformation limits which ranging from A to B “pre-yield zone” in force-deformation relationship for plastic hinges.
- Also, plastic hinges are formed from A to IO “Immediate Occupancy” performance level clarifying that there is no significant damage would occur to structure and the structure can retain its original strength and stiffness.

### 5.2.3 Structure performance points

The structure performance points in X- and Y- directions according to FEMA 440 [3] are shown in Figs. 37 and 38.

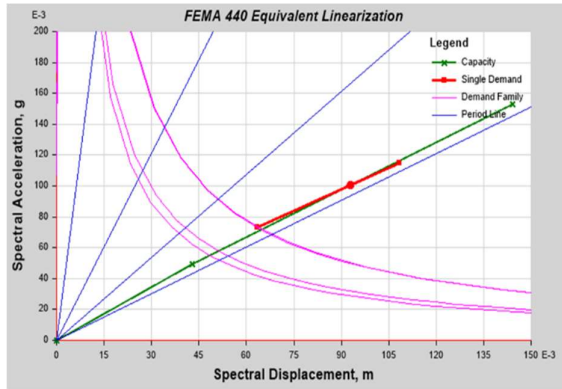


Fig 37: Performance point in X-direction (7562 kN and 168 mm)

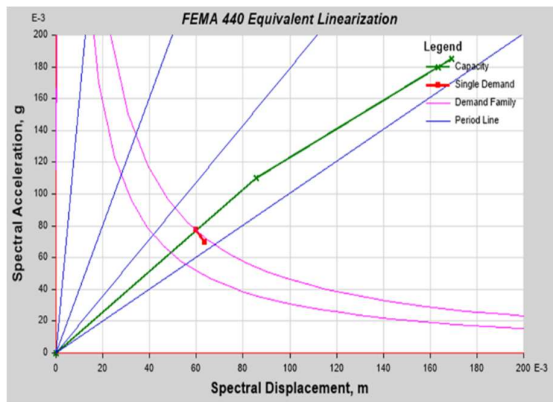


Fig 38: Performance point in Y-direction (7082 kN and 150 mm)

- According to performance points for structure in X-Direction, the structure can sustain seismic base shear equal to 7562 kN which represents about 72% of its ultimate capacity (10483 kN) from pushover analysis. Also, its displacement equals to 168 mm represents about 58% of its ultimate displacements (290 mm).
- According to performance points for structure in Y-Direction, the structure can sustain base shear equal to 7082 kN under prescribed seismic loads which represents about 65% of its ultimate capacity (11001 kN). And, its displacement equals to 150 mm represents about 53% of its ultimate displacements (282 mm).
- It can be concluded that, the obtained results for base shear and displacement are higher than those calculated using the design response spectrum method. Consequently, obtained results according to design code revealing that there are enough strength and displacement reserved at structure performance point.

**5.2.4 Response modification factor**

The R values in both directions calculated using FEMA 356 [1] and ATC 63 [21] are given in Tables 4 and 5, respectively.

**Table 4:** R Values in X- and Y-Directions According to FEMA 356 Method

Direction	X-Direction	Y-Direction
<b>K<sub>i</sub> (kN/m)</b>	47030	50896
<b>K<sub>e</sub> (kN/m)</b>	47030	50896
<b>T<sub>i</sub> (sec)</b>	1.82	1.71
<b>T<sub>e</sub> = T<sub>i</sub> √(K<sub>i</sub>/K<sub>e</sub>) (sec)</b>	1.82	1.71
<b>S<sub>a</sub> (g)</b>	0.1241	0.1312
<b>W (kN)</b>	79606	79606
<b>V<sub>y</sub> (kN)</b>	4501	6089
<b>C<sub>m</sub></b>	1.0	1.0
<b>R = S<sub>a</sub>/(V<sub>y</sub>/W) *C<sub>m</sub></b>	<b>2.20</b>	<b>1.72</b>

**Table 5:** R Values in X and Y – Directions According to ATC 63 Method

Direction	X-Direction	Y-Direction
<b>V<sub>y</sub> (kN)</b>	4501	6089
<b>V<sub>e</sub> (kN)</b>	10483	11001
<b>V<sub>d</sub> (kN)</b>	2701	3654
<b>Δu (mm)</b>	96	120
<b>Δy (mm)</b>	255	274
<b>Ω = V<sub>y</sub>/V<sub>d</sub></b>	1.667	1.667
<b>μ = Δu/Δy</b>	2.665	2.289
<b>Rμ</b>	2.665	2.289
<b>R = Ω. R<sub>μ</sub> R<sub>R</sub> R<sub>ξ</sub></b>	<b>4.45</b>	<b>3.82</b>

- According to calculated response modification factor “R” in both directions, the R values are smaller than or near to allocated in design code ECP 201 [23] “R=5.0” which means that this building has a lower ductility upon on its nonlinear behavior and members ultimate capacity.
- The R values are different in the X- and Y-directions which is more realistic than constant value in both directions mentioned by design code.

**CONCLUSIONS**

The subsequent conclusions can be drawn from the obtained results:

1. The pushover analysis (POA) can anticipate how the strength of the structure will deteriorate as well as where plastic hinges will occur. Additionally, POA identifies structural members that may experience critical phases during an earthquake.

2. For the two study cases designed using response spectrum analysis, the obtained results indicated that the global performance point lies in pre-yield zone or the immediate occupancy (IO) performance level. Consequently, there is an enough safety margin against collapse and adequate strength and displacement are reserved.
3. According to performance points for the structure in study case No.1, the building can sustain seismic base shear equal to 85% and 75% of its ultimate capacity from pushover analysis (POA) in X- and Y-Directions respectively.
4. According to performance points for the structure in study case No.2, the building can sustain seismic base shear equal to 72% and 65% of its ultimate capacity from pushover analysis (POA) in X- and Y-Directions respectively.
5. For study case No.1, the calculated response modification factor "R" values in both directions are greater than the value of 5.5 that allocated in design code UBC97 [22]. This means that this building has a higher ductility and ability for earthquake energy dissipation upon on its nonlinear behavior and its members ultimate capacity.
6. For study case No. 2, the calculated response modification factor "R" values in both directions, the R values are smaller than or near to allocated in design code ECP 201 [23] "R=5.0" which means that this building has a lower ductility upon on its nonlinear behavior and members ultimate capacity.
7. For the study cases presented in this research, the R values are different in the X- and Y-directions which is more realistic than the constant value mentioned by design code.

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