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FORCED-PERFORMANCE CORRELATION FACTOR FOR SEISMIC DESIGN OF MOMENT RESISTING FRAMES

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Abstract: The main objective of seismic engineering is to design the buildings to resist the force generated from ground motion in a safe manner, which prevents the total damage of the building. When the earthquake motion occurs, it was observed that the structure may not totally collapse, but there is a damage level to structures, economic loss because of the inability of the building function, and the cost of the repair was unpredictably high. Existing seismic codes are forced based controlled design, using the base shear concept. The need for changes in the existing seismic design methodology existing in seismic codes has been widely recognized. One of the major developments in seismic design over the past 10 years has been increased emphasis on limit states design, now generally termed Performance-Based Seismic Design (PBSD). Performance-based seismic design is aimed at controlling the structural damage based on precise estimations of proper response of the structure using nonlinear analysis in seismic design. This paper discusses seismic capacity of structure to resist earthquake force for moment resisting frames with medium ductility class using (PBSD) by the capacity spectrum method and obtaining the performance factor (P-Factor) for different levels of damage that can be used in seismic design instead of response modification factor (R-Factor) to overcome the limitation in response modification factor and to represent real performance of the structure.

Index Terms - **Seismic Performance, Nonlinear analysis, Pushover analysis, Capacity Spectrum Method.**

I. INTRODUCTION

Earthquakes are one of the greatest natural hazards which can make uncountable damage to our earth. After several powerful earthquakes that caused major damage in countries with medium and large seismic activity, it has been continuously working on improving methods of design. Thus, in recent years instead of the force-based design method, a performance-based seismic design method has been introduced for the construction of buildings to get a new dimension in earthquake engineering and to evaluate the adequacy of existing buildings under earthquake load. simplified linear elastic method is not sufficient. Current interest in performance-based seismic engineering showed that an inelastic procedure especially pushover analysis is an effective tool to assess the damage vulnerability of buildings. Performance-based seismic approaches were further codified. There are various guidelines have been published on first-generation PBSD methodologies. The most important ones are Vision 2000 (SEAOC 1995), (ATC-40,1996), (FEMA 273,1997), (FEMA 356,2000) (FEMA 440, 2005), (ASCE-41,2006). PBSD guidelines have introduced a comprehensive framework for linear and nonlinear analysis procedures. Nonlinear analysis procedures provide a better perception of inelastic behavior and failure modes of structure during a severe seismic event.

1.1 Response Reduction Factor

The response reduction factor is defined as the factor by which the actual base shear force should be reduced to obtain the design lateral force. It represents the excess of lateral strength that exists in a structure to its designated design strength. the response reduction factor is expressed as a function of various parameters of the structural system, such as strength, ductility, damping, and redundancy. According to ATC-19 response reduction factor is given by the following formula:

$$\mathbf{R = R_s * R_R * R_\mu} \quad \mathbf{(1)}$$

Where Rs is the overstrength factor, Rμ is the ductility factor and R_R is the redundancy factor.

Over strength (R_S): The overstrength factor is a measure of the built-in over strength in the structural system and is obtained by dividing the ultimate base shear (V_u) by the design base shear (V_d)

Ductility Factor (Rμ): It is defined as the capacity to undergo large inelastic deformations without significant loss of strength or stiffness. Ductile structures have been found to perform much better in comparison to brittle structures. Ductility ratio 'μ' is given by $\mu = \Delta u / \Delta y$, where Δu is ultimate deformation and Δy is yield deformation.

Redundancy factor (R_R): The redundant structural system is categorized into the RC structural system with multiple lines of lateral load resisting frames. The redundancy factor for redundant structures is taken as 1.

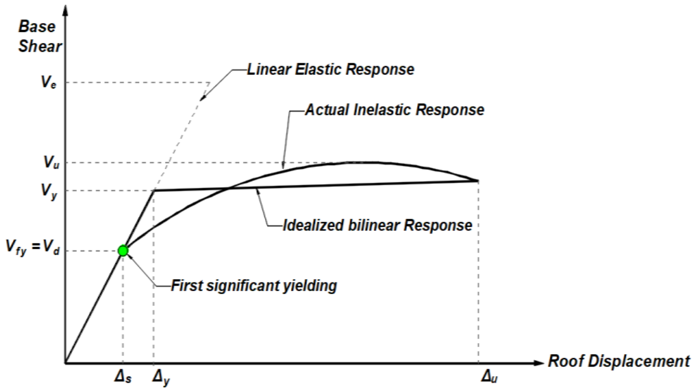

Figure 1 Concept of Response Reduction Factor

1.2 Performance Objectives

Performance-based seismic design explicitly evaluates how a building is likely to perform. A performance objective has two essential parts – a damaged state and a level of seismic hazard. Seismic performance is described by designating the maximum allowable damage state (performance level) for an identified earthquake ground motion. A performance objective may include consideration of damage states for several levels of ground motion. Structural Performance Levels are defined as:

Operational. Facility continues in operation with negligible damage.

Immediate Occupancy Level. Facility continues in operation with minor damage and minor disruption in nonessential services.

Life Safety Level. Life safety is substantially protected, the damage is moderated to extensive. Repairs may be required before the re-occupancy of the building occurs.

Collapse Prevention Level. Life safety is at risk, damage is severe but structural collapse is prevented.

Vision 2000 offers relationships between these hazard and performance levels for various building categories (e.g., hospitals are considered to be critical facilities). This relationship is summarized in the matrix as shown in Figure 2, which indicates that the performance level should be satisfied for the given hazard level and the type of structure.

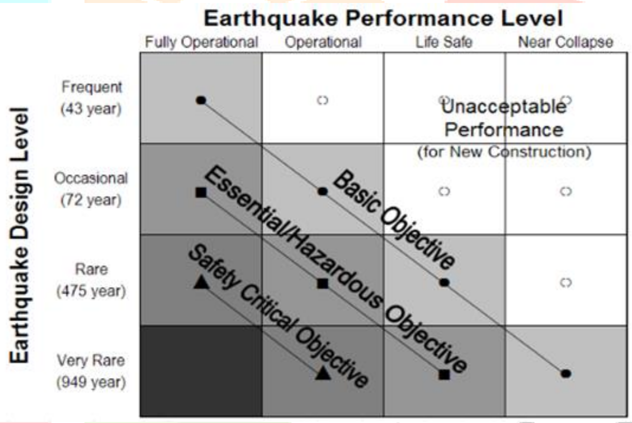

Figure 2 SEAOC Vision 2000 Performance Matrix

1.3 Nonlinear Static Analysis Methodologies for Performance-Based Seismic Design

Capacity Spectrum Method (CSM) in ATC 40

Capacity Spectrum Method (CSM) was first initiated in the 1970s as a rapid evaluation method for assessing the seismic vulnerability of buildings at the Puget Sound Naval Shipyard project. The procedure compares the capacity of the structure (capacity spectrum, i.e. pushover) with the demands of the structure (demand spectrum). The graphical intersection of these two curves is termed as the performance point, which approximates the response of the structures. ATC 40, Seismic evaluation and retrofit of concrete buildings introduced a full guideline for nonlinear static procedures for capacity spectrum method (CSM) for design and evaluation of existing structure. As graphically presented in Figure 3, simplified nonlinear analysis procedures using pushover methods such as capacity spectrum require determination of three essential elements: capacity, demand (displacement) and performance. Each of these elements is discussed briefly below.

Capacity: The definition of capacity is the strength and deformation capacities of the components of the structure. The pushover capacity curve shows approximately how the structure behaves after exceeding its elastic limit.

Demand (Displacement): The displacement demand is an estimate of the maximum expected response of the building during the ground motion.

Performance: Once a capacity curve and demand displacement are defined, a performance check can be done as the performance point is the intersection between the capacity curve and the demand curve. A performance check verifies that structural and non-structural components are not damaged beyond the acceptable limits of the performance objective for the forces and displacements implied by the displacement demand.

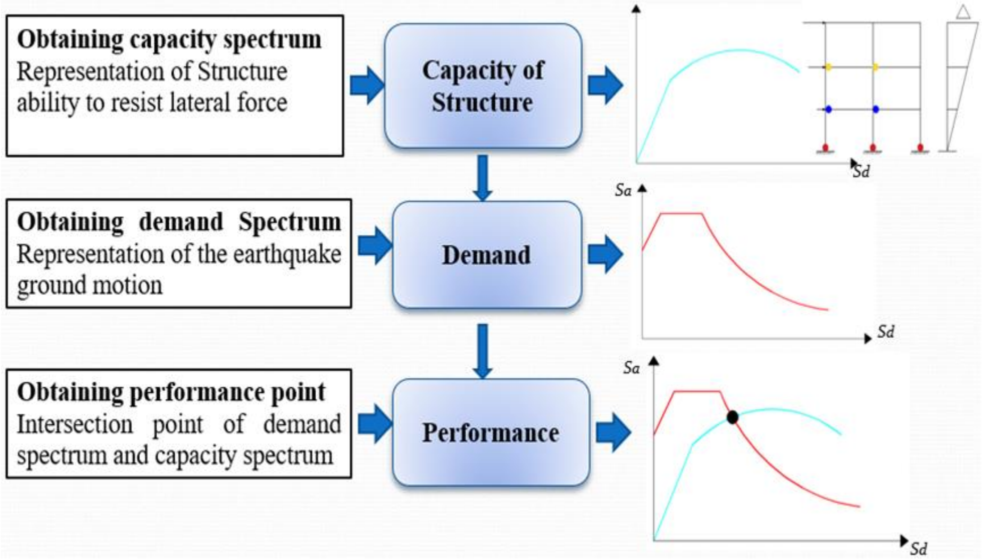

Figure 3 Nonlinear Static analysis procedure for capacity spectrum method

Displacement Coefficient Method (DCM) in FEMA 273 and FEMA 365

FEMA 273 and FEMA 365 guidelines for the seismic rehabilitation of existing building provides a simple method for estimating the target displacement δt. The target displacement refers to the displacement of a characteristic node on the top of a structure, typically on the roof, during a seismic event. It does not require converting the capacity curve into its corresponding spectral coordinates as the capacity spectrum method. The target displacement helps in estimating maximum inelastic deformation demands on a building.

Improvements in the Existing NSPs by FEMA 440

The FEMA 440 (2005) report presented a comprehensive analysis program to develop the improved versions of both the capacity spectrum method (CSM) as well as the displacement coefficient method (DCM). This investigation consisted of determining the peak nonlinear displacements of a large number of SDF systems subjected to a large number of real ground motion records. For the Capacity Spectrum Method (CSM), a more efficient bilinear approximation of the pushover curve is proposed. The expressions to estimate the effective time period and effective viscous damping (for reducing the elastic spectrum to an inelastic demand spectrum) were also improved. Similarly, the FEMA 440 report also provided the improved expressions for the displacement modifying coefficients for the DCM, which were later incorporated in the ASCE/SEI 41-06 (2007) and ASCE/SEI 41-13 (2013).

The NSP in ASCE/SEI 41-06 and ASCE/SEI 41-13

The ASCE-41 (DCM) is based on the improvements to the FEMA-273 and FEMA 356 (DCM) equation by modifying the coefficient used for estimating target displacements. The method has introduced coefficients representing the influence of different soil site classes, which disregards the overestimation of displacements and addresses the effects of strength and stiffness degradation. The improvements have imposed a limitation on the lateral strength to avoid dynamic instability (P–Δ).

The NSP in the N2 method

N2 method is a simple nonlinear method for the seismic performance evaluation of structures was proposed by Peter Fajfar in 1999. The method combines the nonlinear static pushover analysis of the multi-degree of freedom (MODF) model with the response spectrum of an equivalent single degree of freedom (SDOF) system. The approach represents the so-called N2 method formulated in the CSM format. The method provides a visual interpretation of the capacity method with a sound physical basis of an inelastic demand spectra. In the N2 method, N stands for nonlinear analysis and 2 stands for two mathematical methods. The method has been included in the Euro code 8 (EC8:2004).

1.4 Related Work

J.B. Mander, et al. (2001): This study has reviewed from a historical perspective past and current developments in seismic design of structures. Based on the present state-of-the-practice in New Zealand, it is argued that in order to make progress towards the building of seismic resilient communities, research and development activities should focus on two fronts: improved design methodologies and new forms of construction. Improved design methodologies alone will not lead to a significantly superior level of seismic resilient communities, but rather lead to a superior standard of performance-based engineered structures where the post-earthquake outcome will be known with a certain degree of confidence. Therefore, to improve the post-earthquake performance of structures, it will be necessary to develop new forms of construction, which are at least repairable or preferably damage-free. This paper has given two philosophical approaches that are referred to as Control and Repairability of Damage (CARD), and Damage Avoidance Design (DAD).

Andreas J. Kappos and Georgios Panagopoulos (2004): In this paper, a performance-based design procedure is applied for 3D reinforced concrete (R/C) building. Two alternatives analysis methods are suggested, one involving time-history analysis for scaled input motions, and one involving inelastic static (pushover) analysis. A six-storey (R/C) building is first designed to the provisions of the Greek Seismic Code, which is very similar to the Eurocode 8 – ductility class "M" (medium). The proposed method for the performance-based design is then applied to the building and the seismic performance of this building is compared with that of a building designed to current seismic code. It was found that performance-based design leads to better seismic performance than the standard code procedure, and in addition, leads to a more economic design of transverse reinforcement in the members that develop very little inelastic behaviour even for very strong earthquakes.

A. Mwafy, (2013) This paper proposes a simple and theoretically-based approach that utilizes inelastic structural response in design. To describe the approach, the correlation of seismic demands obtained from inelastic and elastic analysis procedures carried out using a wide range of reinforced concrete buildings of different characteristics, ranging from 8 to 60 storey the presented approach enables designers to arrive at a realistic and cost-effective design without compromising safety.

II. Modeling and Analysis

2.1 General

The main objective of the performance-based seismic design of buildings is to avoid total catastrophic damage and to restrict the structural damages caused to the performance limit of the building. For this purpose, Static pushover analysis is used to evaluate the real strength of the structure and obtaining performance factor for moment resisting frames with medium ductility for different performance levels.

2.2 Description of the structural systems considered

A group of Reinforced concrete buildings models of moment resisting frame with medium ductility class is used for the objective of this study that covers different structural heights. Figure 4,5 illustrates the configuration of this group

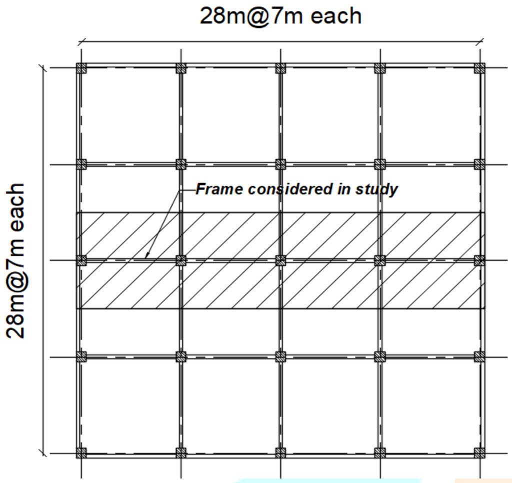

Figure 4 Typical Plan of the study frames

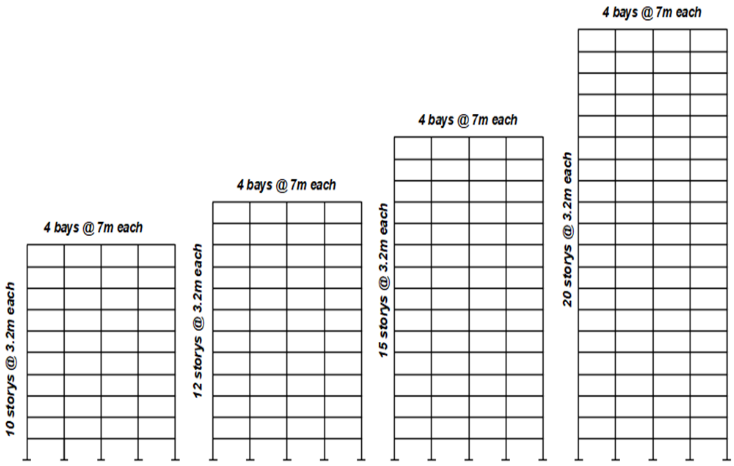

Figure 5 Elevations of the study frames

2.3 Material Properties

The material used for all structural elements is concrete which comply with the Egyptian standard specification.

Concrete:

Compressive strength of concrete cube: Fcu =30 N/mm^2

Density of reinforced concrete =25 N/mm3

Modules of Elasticity =4400√Fcu = 4400√30=24099.8 N/mm^2

Steel:

High strength steel reinforced is used of grade 400/600

Fy=400 N/mm^2

Fu=600 N/mm^2

2.4 Design Loads

2.4.1 Gravity Loads

The first part of gravity loads is the dead load that includes the self-weight of the structure, floor finish, and walls. The second part is the live load that is determined according to the Egyptian Code for calculating loads and forces in structural work related to the main function of the building which is considered as a typical residential building. The values of the dead and live loads are given as follows:

- 16 cm thickness of the reinforced slab
- Flooring Load = 150 kg/cm^2
- Density of brick = 1600
- Live Load = 200 kg/cm^2

2.4.2 Seismic Loads

The seismic loads are calculated according to the Egyptian code simplified modal response spectrum method. Seismic parameters are given as follows:

Soil type: D

Importance Factor γ_I : 1

Seismic Zone: 5A

Design acceleration (a_g) :0.25g

Response Spectrum Type: Type 1

2.5 Dimensional properties

Performance-based seismic design is an iterative process to choose the dimension of structure to reach the required performance level. In this study, the dimensions of the structure are chosen to make the structure safe under vertical loads and studying the performance level then changing the dimensions to reach the desired performance level of the structure.

2.6 Modelling of Structure

ETABS is a widely-used finite-element software package has been prepared by Computers and Structures, for structural analysis and design purpose. It is a fully integrated program for modelling, analysing and designing structures. ETABS analysis machine offers many various features. The most important features for the objective of this research are static and dynamic structural analysis, linear and nonlinear structural analysis, seismic analysis and also static pushover analysis, and a variety of loading choices. This research work adapts ETABS as an aid to perform analysis of structure, it is to be noted that the accurate nonlinear behaviour prediction of the structure also depends upon the precision in describing its component-level performance, the critical aspects of the modelling are described in this section.

2.7 Nonlinear static pushover analysis

A nonlinear pushover analysis is carried out for evaluating the structural seismic response. The pushover analysis consists of the application of gravity loads and a representative lateral load pattern. The lateral loads were applied monotonically in a step-by-step nonlinear static analysis. The applied lateral load was an inverted triangle load pattern in the x-direction representing the forces that would be experienced by the structures when subjected to ground shaking. Under incrementally increasing loads some elements may yield sequentially. Consequently, at each event, the structures experience a stiffness change as shown in Figure 6, where IO, LS, and CP stand for immediate occupancy, life safety, and collapse prevention respectively.

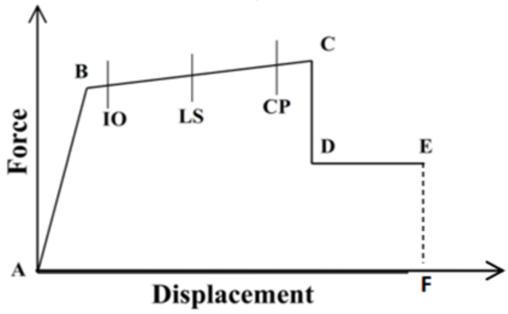

Figure 6 Force-deformation criterion for hinges used in pushover analysis

- Point A: is always the origin.
- Point B: represents the yield point. No deformation occurs in the hinge up to point B .
- Point C: represents the ultimate capacity for the pushover analysis .
- Point D: represents the residual strength for the pushover analysis .
- Point E: represents the total failure. Beyond point E, the plastic hinge will drop a load down to point F directly below point E on the horizontal axis.

III. Results and Discussion

3.1 Introduction

As mentioned earlier various models with different heights are subjected to Pushover analysis. In this section, a comparative study of these curves for different models is carried out to understand the difference in response and behavior of the building with different heights. We get results from the Static pushover curve in terms of base shear, displacement and performance point using the FEMA-440 Equivalent Linearization method which helps to evaluate the Performance factor (P-Factor) for each performance level.

3.2 Nonlinear Static Pushover Analysis

Using ETABS, the non-linear static pushover analysis has been carried out on the dimensional frames. All the beams and columns have been modeled using the two nodded frame elements. In the case of columns, the effect of axial loads on plastic hinges is considered using the P-M-M interaction diagram for each different RC section. No shear hinge formation is considered in this analysis, as various design and detailing provisions specified in ECP eliminate the possibility of such a failure. The pushover curve is generally used to present the output of the non-linear static analysis. Figures 7,8,9,10,11,12,13 represent the pushover curve obtained from the non-linear analysis of the study frames and the bilinear idealization of each curve. .In this study there are some structures did not reach to life safety and collapse prevention levels as the minimum dimensions of the structure that make the structure safe under vertical loads did not allow the structures to high levels of performance as life safety and collapse prevention levels

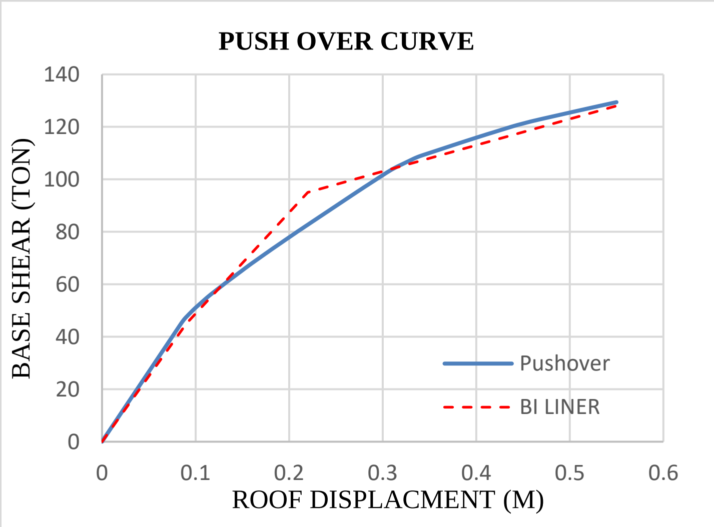

Figure 7 Pushover curve for 10-Storey Frame-IO Level

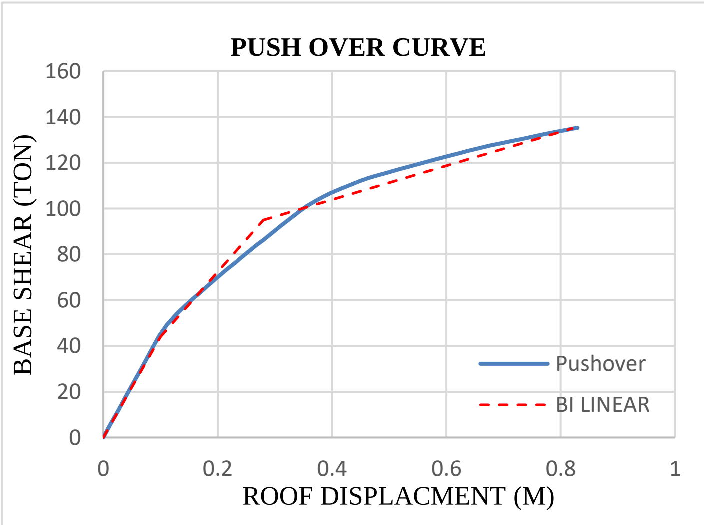

Figure 8 Pushover curve for 12-Storey Frame-IO Level

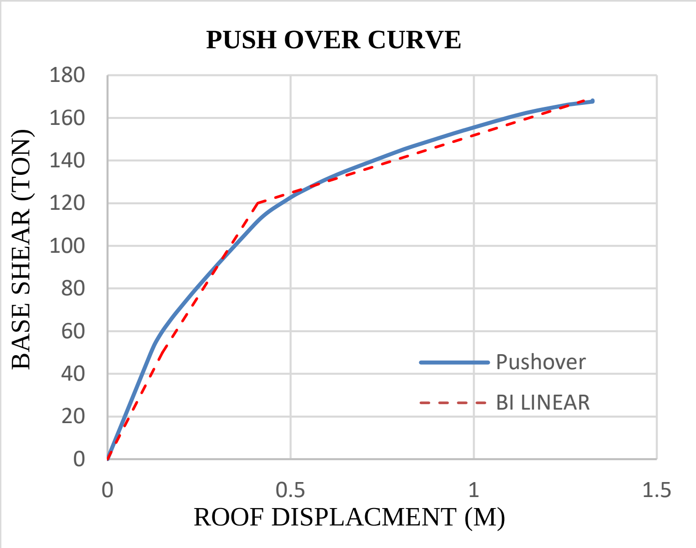

Figure 9 Pushover curve for 15-Storey Frame-IO Level

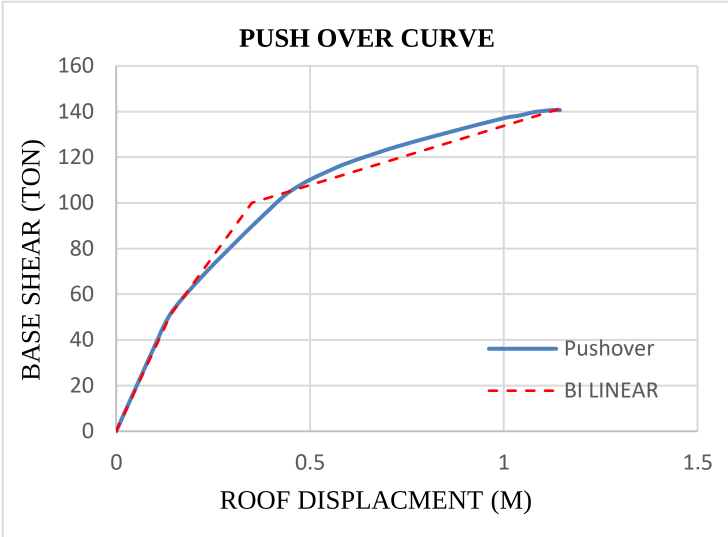

Figure 10 Pushover curve for 15-Storey Frame-LS Level

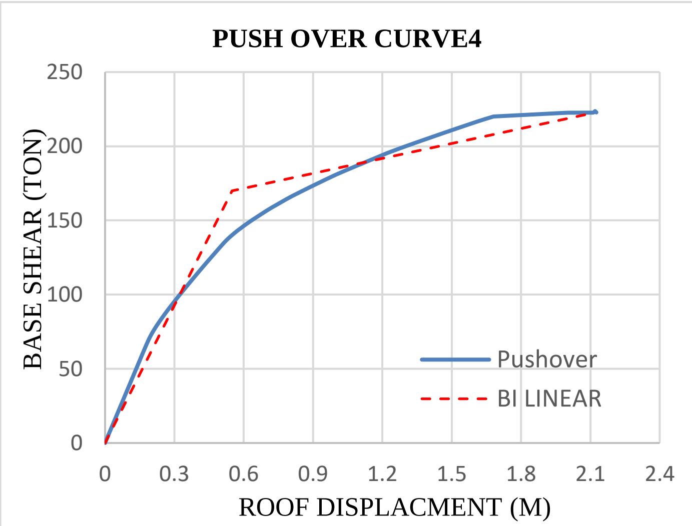

Figure 11 Pushover curve for 20-Storey Frame-IO Level

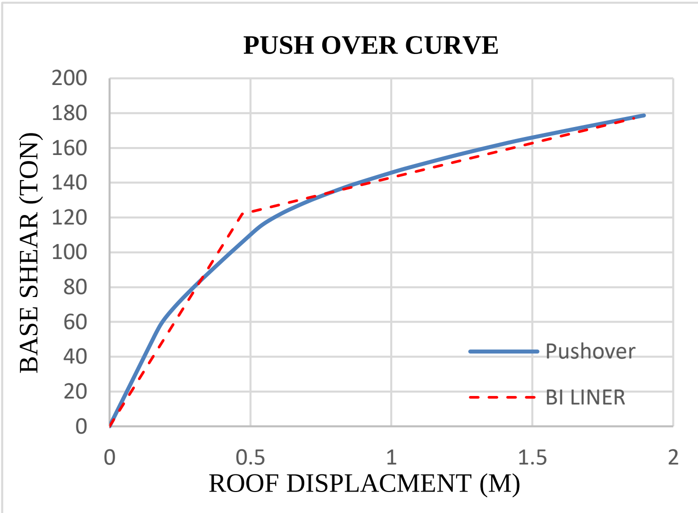

Figure 12 Pushover curve for 20-Storey Frame-LS Level

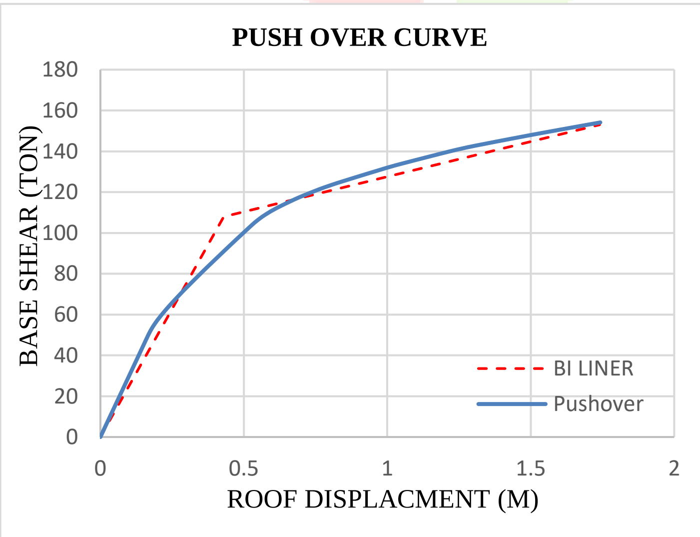

Figure 13 Pushover curve for 20-Storey Frame-CP Level

3.3 Performance Point Determination and level of the structure

Building performance levels can be determined using the capacity spectrum method. The capacity spectrum method allows for a graphical comparison between the structural capacity and the seismic demand. The pushover curve represents the lateral resisting capacity and the response spectrum curve represents the seismic demand. The intersection of the demand spectrum with the nonlinear pushover response is called "Performance Point". A performance check is helpful in determining whether if the structural and non-structural components are not damaged beyond the acceptable limits. Depending on the position and state of the performance point (with respect to the actual pushover curve), the analyst may decide on how safe or vulnerable the structure is and where possible strengthening should be performed. After pushover analysis, the demand curve and capacity curves are obtained to get the performance point of the structure. The performance point is obtained as by capacity spectrum method. Figures 14,15,16,17,18,19,20 show the ADRS spectra (Acceleration -Displacement Response Spectrum) and the performance point of the study frames

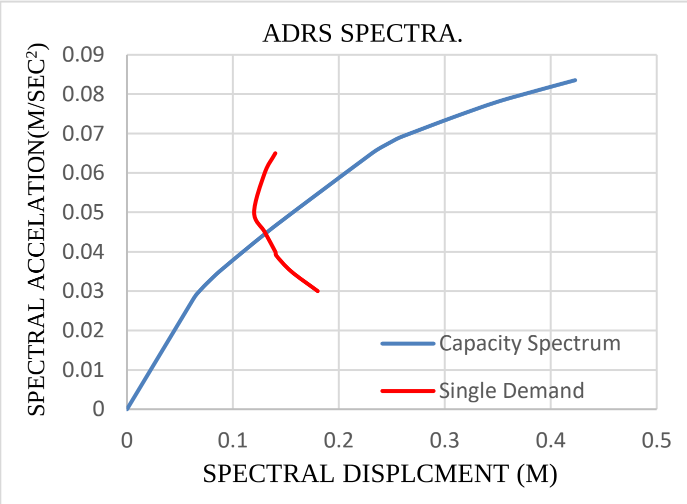

Figure 14 ADRS SPECTRA. for 10-Storey Frame-IO Level

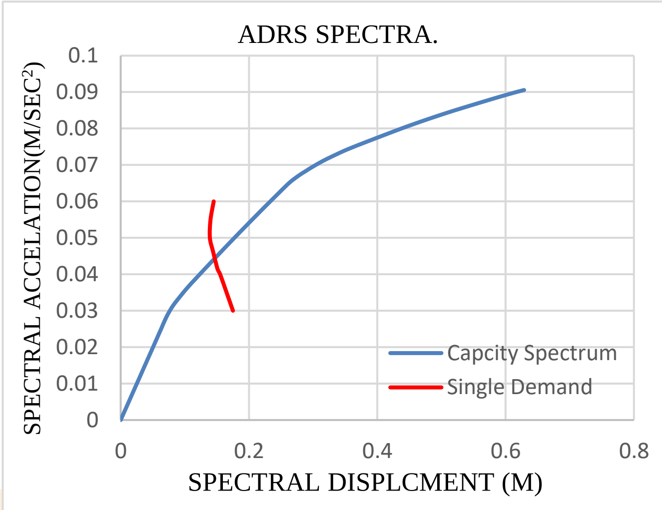

Figure 15 ADRS SPECTRA. for 12-Storey Frame-IO Level

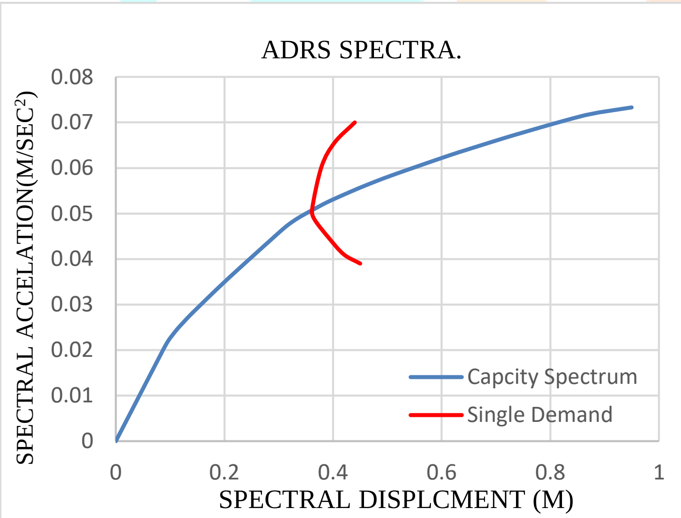

Figure 16 ADRS SPECTRA. for 15-Storey Frame-IO Level

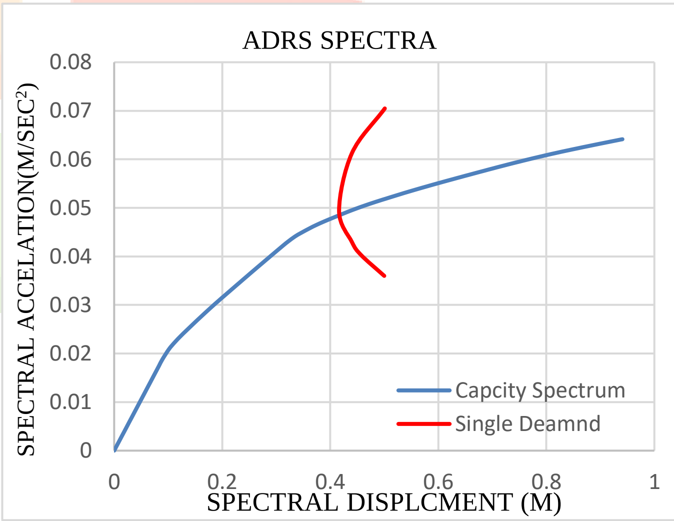

Figure 17 ADRS SPECTRA. for 15-Storey Frame-LS Level

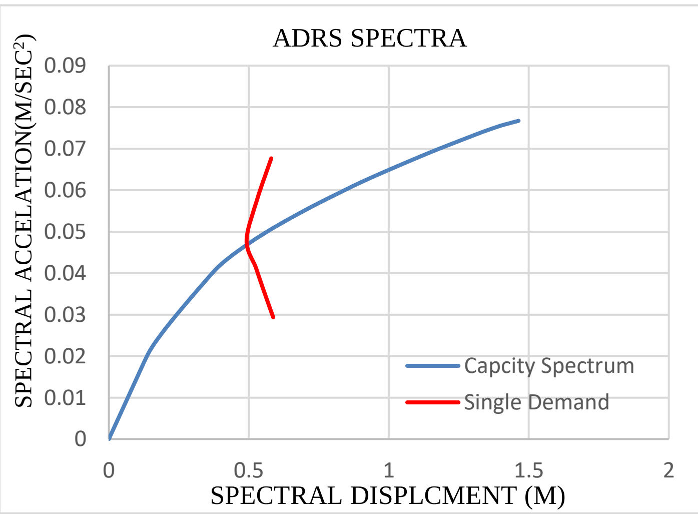

Figure 18 ADRS SPECTRA. for 20-Storey Frame-IO Level

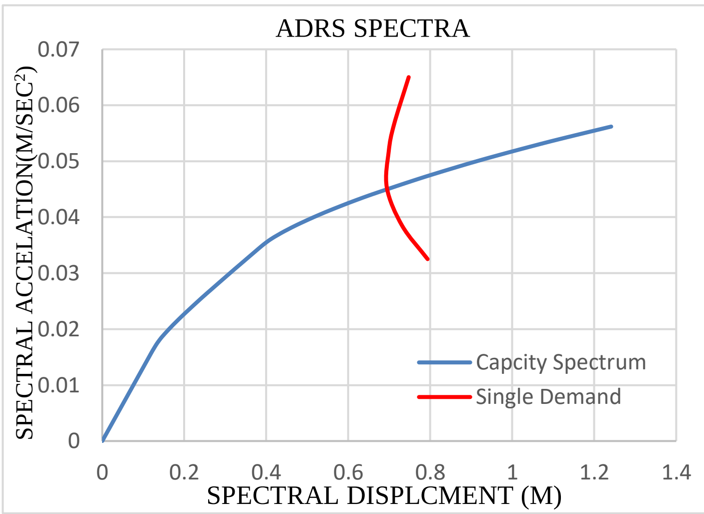

Figure 19 ADRS SPECTRA. for 20-Storey Frame-LS Level

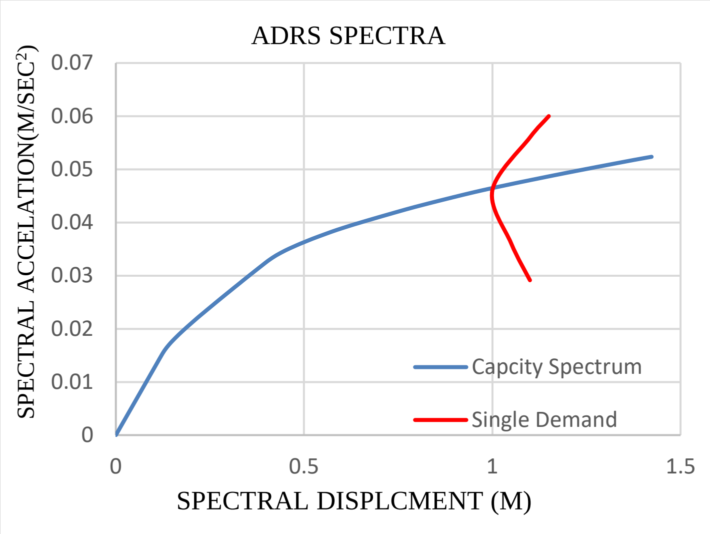

Figure 20 ADRS SPECTRA. for 20-Storey Frame-CP Level

As per the ATC-40, the lateral displacement at the performance point is to be checked against the displacement limits to estimate the damage level as specified in Table 1 according to ATC-40. The damage that occurred in a building is categorized into 4 performance levels namely immediate occupancy, damage control, life safety, and structural stability. For Structural stability, the maximum total drift in story i at the performance point should not exceed the quantity $0.33 Vi/Pi$, where Vi the total calculated shear force in story and Pi is the total gravity load (i.e. dead plus likely live load) at story i. For example, a structure in the Immediate occupancy performance level means less physical damage has occurred in the structure as compared to the structure in the Structural stability performance level. Higher performance means less physical damage to the structure after an earthquake. The performance of all the four frames has been computed. For checking the ADRS of the buildings ETABS has been used and the effective time period of the structure has been obtained. Hence the total drift of the frame is considered in order to check with the displacement limits corresponding to the effective time period (T_{eff}) which represents the time period of the structure after the formation of plastic hinges and the performance level is thus obtained. The total drift of the structures is tabulated in Table 2.

Table 1: Drift Limits

Drift Limitation	Level of Structure Performance			
	Immediate Occupancy	Damage Control	Life Safety	Structural Stability (Collapse Prevention)
Max. total drift	1%	1%-2%	2%	$0.33\frac{Pi}{Vi}$

Table 2: Total drift of structures.

Frame Type	Height (m)	Performance Level	T_{MODE1}	T_{eff}	Total drift
10-Story	32	IO	2.90	3.30	0.18
12-Story	38.4	IO	3.10	3.44	0.20
15-Story	48	IO	4.20	6.00	0.40
		LS	4.40	6.40	0.49
20-Story	64	IO	5.20	7.00	0.58
		LS	5.50	7.40	0.68
		CP	5.70	8.50	0.80

3.4 Computation of Performance Seismic Design factor

The objective of this study to get a performance factor (P-Factor) for each level of performance that could be used instead of a response reduction factor specified in seismic codes to represent the actual response of the structure. From the pushover curves and ADRS curves, the base shear at the performance point is obtained that represents the actual base shear of structure. The performance factor values are obtained from the computed base shear at performance level and the base shear obtained from seismic code. Tables 3,4 presents the component-wise calculation of the (P- Factor) for the building frames.

Table 3: Pushover parameters for the study frames

Frame Type	Performance Level	Δu	Δy	V_u	V_d	$\mu=\frac{\Delta u}{\Delta y}$	$\Omega=\frac{Vu}{Vd}$
10-Story	IO	0.54	0.23	128	48	2.35	2.6
12-Story	IO	0.8	0.29	136	60	2.75	2.28
15-Story	IO	1.25	0.43	168	71	2.90	2.36
	LS	1.14	0.35	139	60	3.25	2.26
20-Story	IO	2.00	0.60	218	85	3.30	2.56
	LS	1.8	0.48	172	70	3.75	2.45
	CP	1.7	0.43	152	63	3.95	2.41

Table 4: Components of the performance factor for the study frames

Frame Type	Performance Level	Rs	Rµ	R_R	P-Factor=RsXRµXR_R
10-Story	IO	2.60	2.35	1	6.11
12-Story	IO	2.28	2.75	1	6.27
15-Story	IO	2.36	2.90	1	6.84
	LS	2.26	3.25	1	7.34
20-Story	IO	2.58	3.30	1	8.44
	LS	2.45	3.75	1	9.18
	CP	2.41	3.95	1	9.51

3.5 Computation of correlation factor (ρ) between Forced based design and performance-based design

Correlation factor could be obtained between forced based design and performance-based design that represent the ratio between (P-factor) and codal response reduction factor (R=5.00). Table 5 presents the correlation factor for different levels of structure

Table 5: Correlation Factor

Frame Type	Performance Level	P-Factor	R_{code}	Correlation factor ρ=P-Factor/Rcode
10-Story	IO	6.11	5.00	1.22
12-Story	IO	6.27	5.00	1.25
15-Story	IO	6.84	5.00	1.36
	LS	7.34	5.00	1.46
20-Story	IO	8.44	5.00	1.68
	LS	9.18	5.00	1.83
	CP	9.51	5.00	1.90

3.6 Effect of number of stories on the value of 'Performance Factor'

The performance factor has been calculated for the four moment resisting frames with medium ductility class with different numbers of stories. With the increase in height, the performance factor increases 'P-factor' values obtained are in the range of 6 to 8.5 for frames for immediate occupancy level and in the range of 7.34 to 9.18 for life safety level and in range of 9.51 for collapse prevention level, these values are greater than the value provided in Egyptian code for such frames which is equal 5.00 Figure 21 and show the variation in the 'P' factor with increase in the number of stories.

3.7 Variation in the ductility factor.

It is observed that the ductility factor increases with the increase in height, for the 12-storey building exceeded the value for 10-storey building by 14.5% and that for 15-storey building it is exceeded by 5.18% when compared with a 12-storey building. Figure 22 shows the graphical representation of the variation of the ductility factor

3.8 Variation in the overstrength factor.

The excess strength of the structure because of load factors in various design load combinations and reduced design strength of materials by a factor of safety is taken into account by the strength factor. It has been observed that there is no trend of increasing strength factor as ductility factor with the increase of height. However, it is seen that some shorter frames show higher over-strength values as compared to the taller frame. Figure 23 shows the graphical representation of the variation of strength factor values with the number of stories for all the four frames.

3.9 Variation of 'Performance Factor' with parodic time

Figure 24 shows the graphical representation of the performance factor values with the parodic time of the first mode.

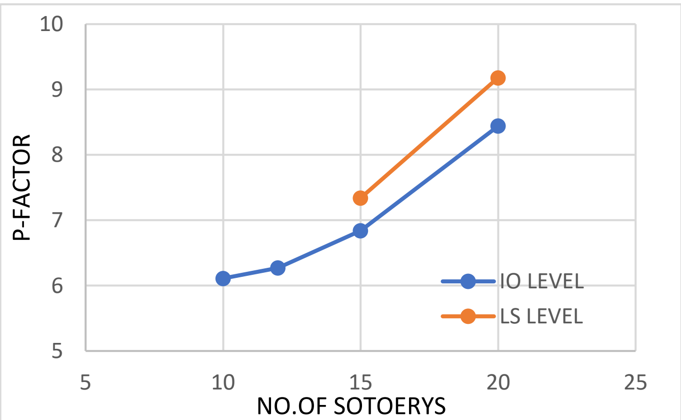

Figure 21 Effect of number of storeys on the value of 'Performance Factor'

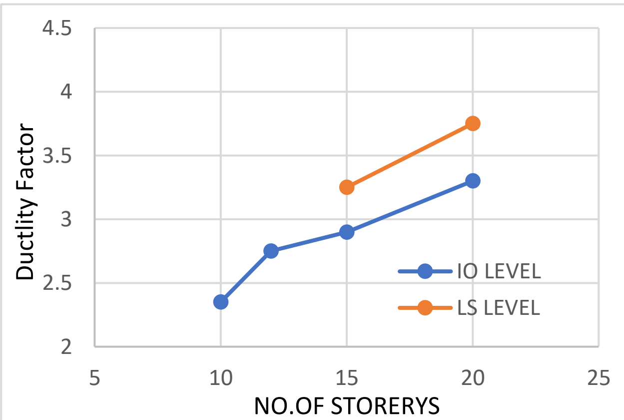

Figure 22 Variation in the ductility factor

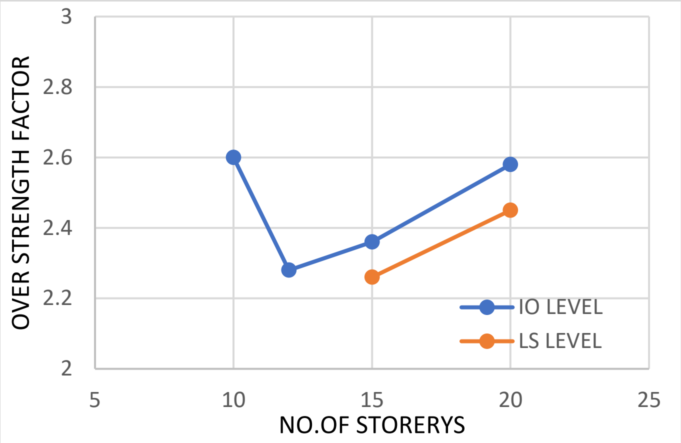

Figure 23 Variation in the over strength factor.

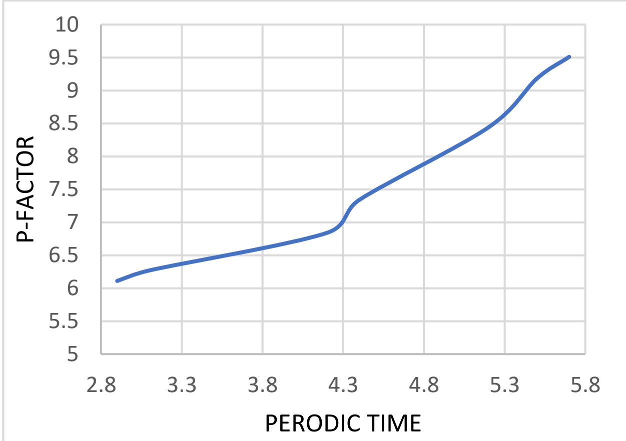

Figure 24 Variation of 'Performance Factor' with parodic time

3.10. Conclusion

1. The Egyptian design code does not specify any mathematical basis for the response reduction factor. A single value of R for all the buildings of a given frame type, irrespective of the plan and vertical geometry cannot be justified.
2. There is a significant difference in the values of response reduction specified in the codes and that obtained from the analysis. In present considered structural system the response reduction factor value is on the higher side as compared to the Egyptian code (ECP-201) value. Egyptian code overestimates the value of the design base shear. So, the R-value suggested by (ECP-201) is on a far more conservative side.
3. As compared to taller frames the shorter frames show higher over strength values.
4. As the height of the structure increases the ductility factor and performance factor also shows an increasing trend.
5. The actual value of (R) are subjected to be different in real life than what is computed here, because of various reasons like irregularity in dimensions leading to torsional effects, lack of quality control and poor workmanship during construction, not following the ductile tensile requirements as specified in the codes, etc.
6. An accurate estimation of the fundamental period is necessary for estimating a realistic R-value for a structure.
7. The conventional code-based method has many deficiencies as a higher mode contribution, stiffness degradation, and the period elongation is not considered.
8. The conditions of the present study are limited by the fact that only a single plan configuration (without asymmetry) in one single seismic zone has been considered. The different parameters used in the work presented are considered to be deterministic, although, in reality, their statistical variations are significant enough requiring a reliability-based framework for the study.

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