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Performance-Based Seismic Analysis of RC Structures with Vertically Variable Lateral Load Resisting System

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Abstract: The choice of an appropriate response reduction factor (R) in the seismic design of a reinforced concrete building is crucial to the building's seismic response. Static non-linear pushover analysis is used to achieve this target by figuring out the building's actual stiffness. Sometimes designing a building with a vertical combination of several lateral force-resisting systems is important in structural engineering. For example, the structure's upper portion could be a shear wall or braced frame, and the bottom could be a rigid frame for example. This study uses finite element models with varying story numbers and lateral force resisting system configurations to examine the impact of vertically variable lateral load resisting systems through utilizing Performance-based seismic design (PBSD), which gained popularity over the previous few years and by introducing response modification factor (R). For the objective of this study, three-dimensional (3D) models are used for two different residential reinforced concrete building groups that cover different heights and lateral system configurations, group (1) includes 10 buildings with different numbers of storeys (10,15,20) floors with both Moment-Resisting Frames at some floors and Shear Walls (at X global direction) at other ones, group (2) includes 10 buildings with different numbers of storeys (10,15,20) floors with both Moment-Resisting Frames at some floors and Shear Walls (at both X global direction and Y global direction) at other ones. The study concluded that configuration of the lateral system in the vertical direction is sensitive and has a significant impact on the response reduction factor.

Index Terms – Performance-Based Design, Non-Linear Static Pushover Analysis, Response Modification Factor, Shear Walls, RC Moment Resisting Frames.

1.INTRODUCTION

Preliminary seismic design techniques based on performance were established in the past after much research. Due to severe earthquakes in seismically active countries, efforts have been made to enhance design methodologies. Recently, a performance-based seismic design method has been offered as an alternative to the force-based design method. It is discussed whether the various methodologies recommended in this study and the current seismic code approaches are appropriate for use in the initial building design phase of future performance-based building designs. A design philosophy known as performance-based seismic design (PBSD) defines the input criteria as performance targets at different degrees of seismic hazard. This definition is reliable. In terms of functionality, safety, or cost, "performance" refers to the structure's capacity to meet stakeholder targets as well as its intended use. Performance in structural terms is typically used to characterize damage levels. Regarding first-generation PBSD techniques, numerous guidelines have been released such as (ATC-40, 1996), (ATC-19, 1996), (FEMA 356, 2000), (FEMA 440, 2005), (ASCE 41-13, 2013). An advanced degree of analysis is required for performance-based seismic design. PBSD techniques provided a thorough foundation for both linear and non-linear analysis techniques. For the objective of this study, three-dimensional (3D) models are used for two different residential reinforced concrete building groups that cover different heights and lateral system configurations, group (1) includes 10 buildings with different numbers of storeys (10,15,20) floors with both Moment-Resisting Frames at some floors and Shear Walls (at X global direction) at other ones, group (2) includes 10 buildings with different numbers of storeys (10,15,20) floors with both Moment-Resisting Frames at some floors and Shear Walls (at both X global direction and Y global direction) at other ones. The study concluded that configuration of the lateral system in the vertical direction is sensitive and has a significant impact on the response reduction factor.

1.1 Performance Levels

According to (FEMA 356, 2000), the building performance level is the result of combining the structural and nonstructural performance levels, and it describes the approximate limiting levels of structural and nonstructural damage that may be expected of buildings rehabilitated to the levels defined in the standard as three performance levels as follows:

Immediate Occupancy: the state of damage following an earthquake where virtually little structural damage has happened. The building's main force-resisting systems, both vertical and lateral, still have almost all its pre-earthquake rigidity and strength. Structural damage carries a very minimal chance of life-threatening injury, while minor structural repairs could be necessary.

Life Safety: refers to the state of the structure after an earthquake where there has been severe damage to the structure but there is still some safety margin against a partial or complete structural collapse. Although several structural parts and sections are seriously damaged, there aren't any significant risks of falling debris as a result.

Collapse Prevention: denotes the condition of the building following an earthquake, when it is in danger of collapsing whole or partially. There has been substantial damage to the structure, which may include a notable deterioration in the lateral force resisting system's strength and stiffness.

1.2 Techniques for Nonlinear Static Analysis for Performance-Based Seismic Design

Capacity Spectrum Method (CSM) in (ATC-40, 1996)

(ATC-40, 1996) presented the Capacity Spectrum Method, a nonlinear analysis technique that defines two essential elements: capacity and demand. Demand (Demand Spectrum) is a picture of the ground motion during an earthquake. On the other hand, capacity (Capacity Spectrum) shows how well the structure can withstand an earthquake. The performance point, which characterizes the behavior of the building, is defined as the junction of two curves. (ATC-40, 1996) outlined comprehensive guidelines for the capacity spectrum method (CSM) and stressed its application to seismic design and existing structure evaluation. The capacity spectrum curve is created by converting the pushover curve to the capacity spectrum curve, where base shear (V_b) becomes spectral acceleration (S_{ai}), and roof displacement (Δ_{roof}) becomes spectral displacement (S_{di}). Conversely, the effective damping of the comparable linear system reduces the 5% damped acceleration response spectrum. Additionally, the decreased response spectrum is transformed into a "demand spectrum" using the ADRS format. The performance point, or approximately expected maximum displacement and base shear based on the chosen demand on the structure, is the intersection point of the capacity and demand spectrums, which are both plotted on the same ADRS format.

Displacement Coefficient Method (DCM) in (FEMA 356, 2000)

The displacement coefficient method was developed by promoting the idea that the largest inelastic displacement can be roughly estimated by multiplying the peak displacement as if the structure remains in an elastic state by modification factors. This method is an attempt to create more precise non-linear static procedures.

Improvement in nonlinear static seismic analysis procedure (FEMA 440, 2005)

The (FEMA 440, 2005) created a comprehensive analysis process study to enhance the displacement coefficient methodology (DCM) and capacity spectrum methodology (CSM) procedures.

For the capacity spectrum method (CSM), A more efficient bilinear idealization approximation of the pushover curve was presented in FEMA 440. The terms used to estimate the effective viscous damping and the effective time period (for converting the demand spectrum with the inelastic state and degrading the elastic spectrum) were improved. Equations from statistical studies are used to produce the damping characteristic, and the structure's equivalent strength stiffness is calculated from its effective periodic time.

For the displacement coefficient methodology (DCM), FEMA 440 developed terms of the displacement corrections coefficients using sophisticated formulas for the (DCM), A horizontal dynamic instability check is used to replace the coefficient of the P-Δ effects by determining the highest value of lateral stiffness strength.

1.3 Effect of Vertically Variable Lateral Load Resisting System

Sometimes designing a building with a vertical combination of several lateral force-resisting systems is important in structural engineering. For example, the structure's upper portion could be a shear wall or braced frame, and the bottom could be a rigid frame for example.

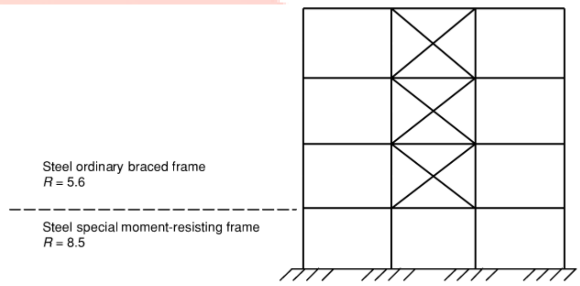
The Uniform Building Code (UBC, 1997) introduced requirements shall be satisfied where combinations of lateral structural systems are incorporated into the same structure. The value of R used in the design of any story shall be less than or equal to the value of R used in the given direction for the story above. Structures may be designed using the procedures of this section under the following conditions:

I. The entire structure is designed using the lowest R of the lateral-force-resisting systems used, or
II. The following two-stage static analysis procedures may be used for the following structures:

Structures having a flexible upper portion supported on a rigid lower portion where both portions of the structure are considered separately can be classified as being regular, the average story stiffness of the lower portion is at least 10 times the average story stiffness of the upper portion and the period of the entire structure is not greater than 1.1 times the period of the upper portion considered as a separate structure fixed at the base.

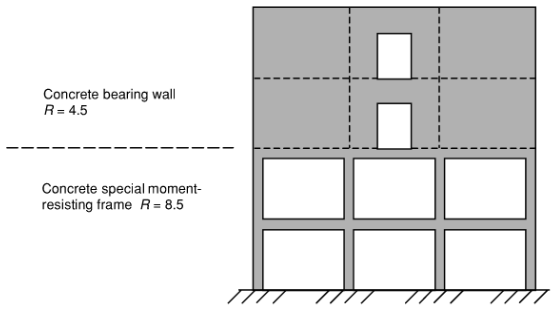
a. The flexible upper portion shall be designed as a separate structure, supported laterally by the rigid lower portion, using the appropriate values of R and ρ.
b. The rigid lower portion shall be designed as a separate structure using the appropriate values of R and ρ. The reactions from the upper portion shall be those determined from the analysis of the upper portion amplified by the ratio of the (R/ ρ) of the upper portion over (R/ ρ) of the lower portion.

For more clarification (SEAOC (Structural Engineers Association of California), 2015) provides examples for determining R-value for structures with a combination of structural systems in the vertical direction.

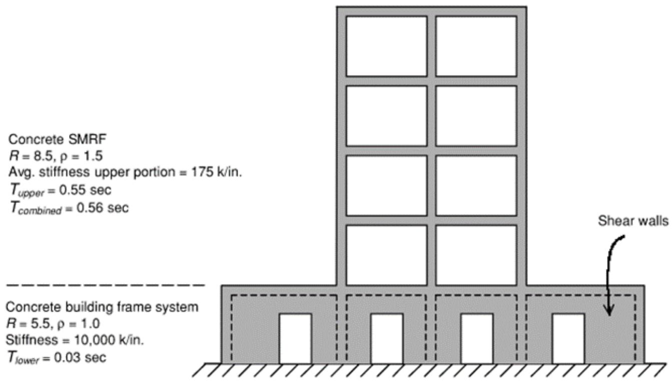


i. Steel ordinary braced frame over steel SMRF.
The rigid system is above the flexible system, Item (II) cannot be used.
Therefore, under Item (I), the entire structure must use R=5.6

ii. Concrete bearing wall over concrete SMRF.
The rigid portion is above the flexible portion, Item (II) cannot be used.
Therefore, under Item (I), the entire structure must use R=4.5

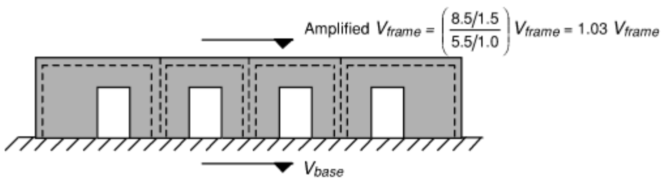


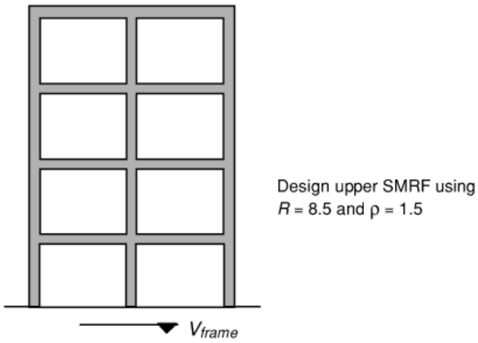
iii. Concrete SMRF over a concrete building frame system.
This is a vertical combination of a flexible system over a more rigid system.
Under Item (I), the two-stage static analysis may be used, provided the structures conform to the following criteria:
a. Flexible upper portion supported on rigid lower portion. o.k.
b. Average story stiffness of the lower portion is at least 10 times the average story stiffness of the upper portion.



10,000 k/in. > 10(175) = 1750 k/in. o.k.
The period of the entire structure is not greater than 1.1 times the period of the upper portion.
0.56 sec < 1.1(0.55) = 0.61 sec o.k.

So, Item (II) can be used:





Design the lower portion of the building frame system for the combined effects of the amplified V_{frame} force and the lateral forces due to the base shear for the lower portion of the structure (using R=5.5 and ρ=1.0 for the lower portion).
So: V_{base} = (Amplified V_{frame}) + (V_{lower})

1.4 Related Work

(Divya & Murali, 2022): performed comparative analysis of the behavior of horizontal and vertical irregular buildings with and without using shear walls. The purpose of this research is to determine which irregularity configuration structure, for the models included in the study, performs well under lateral loads. The studied buildings consisted of ground + 15 multi-story commercial structures maintaining the same plan dimensions and story height for irregularities in mass, stiffness, vertical geometry, plan (horizontal), and mass with and without shear walls. A 30 m × 30 m plot is taken into consideration, with a 5 m column-to-column distance maintained in both the × and Y directions. Four different forms of irregular models, both with and without shear walls, were modeled within this 30 m x 30 m design. The study concluded that shear walls improved the overall performance of irregular building models under lateral forces by around 60%–70%. The building's story displacement and story drift have decreased by more than 30%, increasing the structure's rigidity to withstand lateral forces. Shear walls contribute to a building's reduced structural requirements, which lowers the mass of the building overall and helps reduce story shear. When compared to an irregular building without shear walls, the irregular structure with shear walls is more cost-effective.

(Mabrouk et al., 2020): investigates the performance evaluation of structures that are designed by the existing code requirements. Also, another attempt is made to use the "P" factor in the "R" factor's development to account for varying performance levels. The performance factor “P” will be computed for a variety of RC moment frame structures, from regular to irregular. For this study, a variety of medium- and high-rise structures with varying heights have been chosen. On conclusion, the study shows that (ECP-201, 2012) uses a conservative R-factor for limited ductility RC moment frames, which results in an immediate occupancy level. And in comparison, to all other irregular models, regular models have demonstrated greater values for the maximum ratio of base shear and structure weight; this can be explained by their higher P-Factor and superior performance outcomes. The study has proved that, despite their detrimental effect on the structure's performance, the code assumptions for 'R' value of 5.0 for RC moment frame structures with limited ductility class remain valid. Performance study of various buildings has been accomplished and demonstrates that a building's drift value increases along with a drop in base shear and weight ratio performance points as structure height increases. Different irregular structures' performance points have demonstrated a higher reduced value when compared to regular ones, indicating their detrimental effects on the structure's performance.

(Ahmed et al., 2021): performs pushover analysis to look at how moment-resistant frame (MRF) systems affect the R values of RC structures when there is vertical geometric irregularity. Numerous situations of vertical irregularity, including setbacks and soft stories are handled in this way. Two types of vertical irregularities are considered in this research. Firstly, stiffness irregularity (soft story), Secondly, symmetric, and asymmetric set-back irregularities are studied with stiffness irregularity. The study concluded that for vertically uneven buildings, the seismic force reduction using the R factor becomes a highly misleading value. In fact, because of the obvious overstrength and general ductility fault, the computed R-value is decreased by 20–40%. Also, Vertical irregular buildings suffer from less top displacement and base shear capacity with 10% and 30%, respectively for soft-story irregular buildings compared with regular ones. Moreover, about 20% and

40% reductions occur in top displacement and base shear capacity, respectively, for buildings with combined setbacks and soft story vertical irregularities.

2. MODELING

2.1 Verification

Analytical results of moment-resisting RC frames extracted by (Yosry et al., 2020), were used to verify the analytical models. In his study, to overcome the limitations of the response modification factor and to accurately represent the structural performance, Yosry has conducted an analytical study to obtain the performance factor (P-Factor) for various levels of damage that can be used in seismic design instead of the response modification reduction factor (R-Factor). Two-dimensional (2D) models covering varying heights and bayes are utilized for the study's purpose, of two distinct residential reinforced concrete structure groups.

Group (1): According to ECP-201, the models of Group (1) consist of six different moment-resistant frames with medium ductility. These frames are made of concrete and consist of four bays, with 5, 10, 12, 15, 18, and 20 stories. The elevations of the frames are regular. This group's configuration is shown in Figure 1.

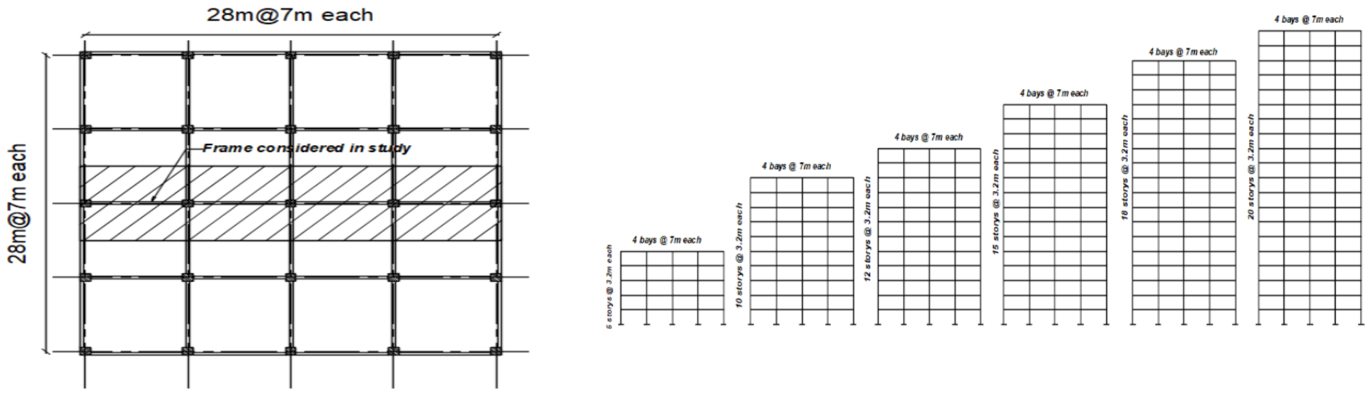


Figure 1 Group 1 configuration

Group (2): According to ECP-201, the models of Group (2) consist of six different moment-resistant frames with medium ductility. These frames are made of concrete and consist of six bays, with 5, 10, 12, 15, 18, and 20 stories. The elevations of the frames are regular. This group's configuration is shown in Figure 2.

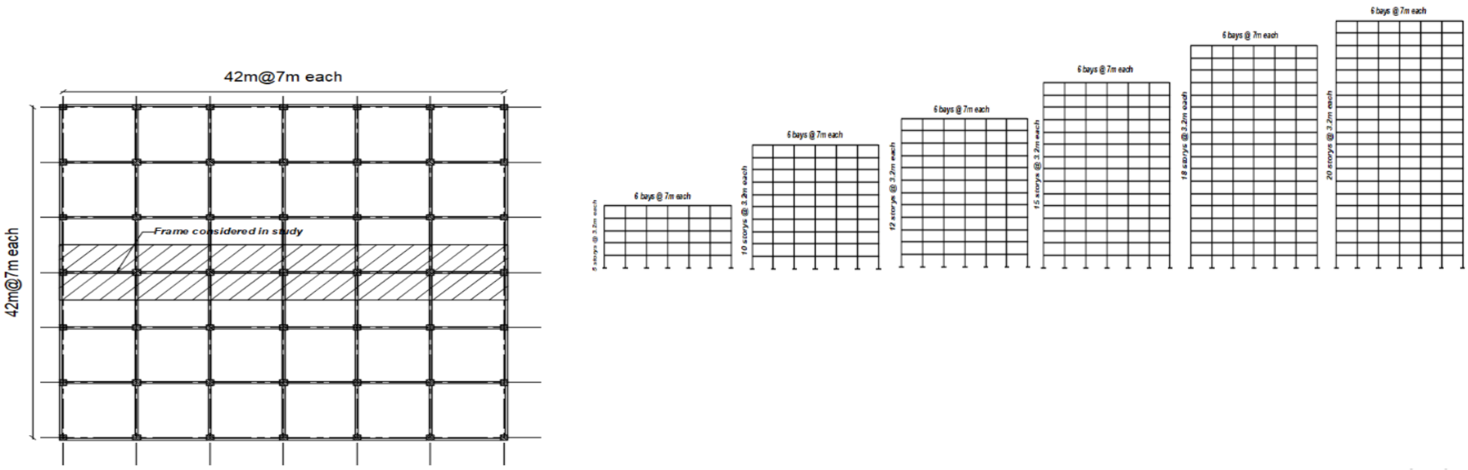


Figure 2 Group 2 configuration

First, finite element ETABS models were generated for moment-resisting RC frames according to the generated analytical models by (Yosry et al., 2020), the columns and beams were defined with the reinforcing details. The frames are subjected to a nonlinear push force load case and then, verification was made for the models introduced by Yosry, where the maximum error was 10.18% as shown in Table 1 and Table 2.

R (Generated ETABS Model)	P (Thesis Model)	Error %
4.98	4.62	7.87%
6.42	5.97	7.50%
6.52	6.32	3.16%
6.18	6.84	-9.66%
7.82	7.34	6.51%
7.29	7.26	0.41%
7.89	8.06	-2.15%
8.92	8.10	10.18%
7.60	8.44	-10.00%
9.88	9.03	9.46%
10.15	9.30	9.13%

Table 1 Performance factor verification for Group 1

R (Generated ETABS Model)	P (Thesis Model)	Error %
4.24	4.71	-10.03%
6.58	6.05	8.80%
6.84	6.49	5.36%
6.56	7.27	-9.81%
8.07	7.38	9.29%
7.83	7.67	2.05%
7.87	8.04	-2.11%
9.02	8.24	9.51%
8.06	8.55	-5.77%
10.32	9.39	9.90%
10.25	9.42	8.82%

Table 2 performance factor verification for Group 2

2.2 Modeling and Analysis

For the objective of this study, three-dimensional (3D) models are used for two different residential reinforced concrete building groups that cover different heights and lateral system configurations. They were first designed under vertical loads only.

Group (1): Models of Group (1) include 10 Buildings with different numbers of stories (10,15,20) floors with both Moment-Resisting Frames at some floors as shown in Figure 3 and Shear Walls (at X global direction) at other ones as shown in Figure 4

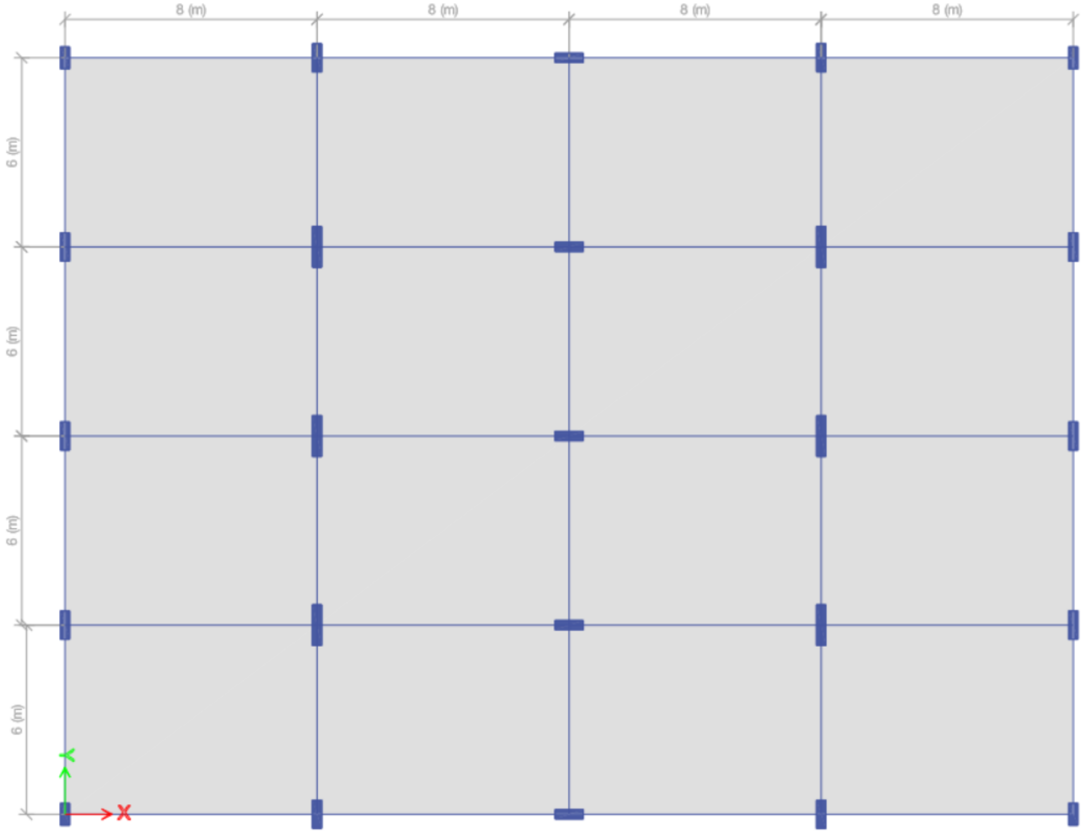


Figure 3 Moment Resisting Frame - Lateral Load System for Group 1

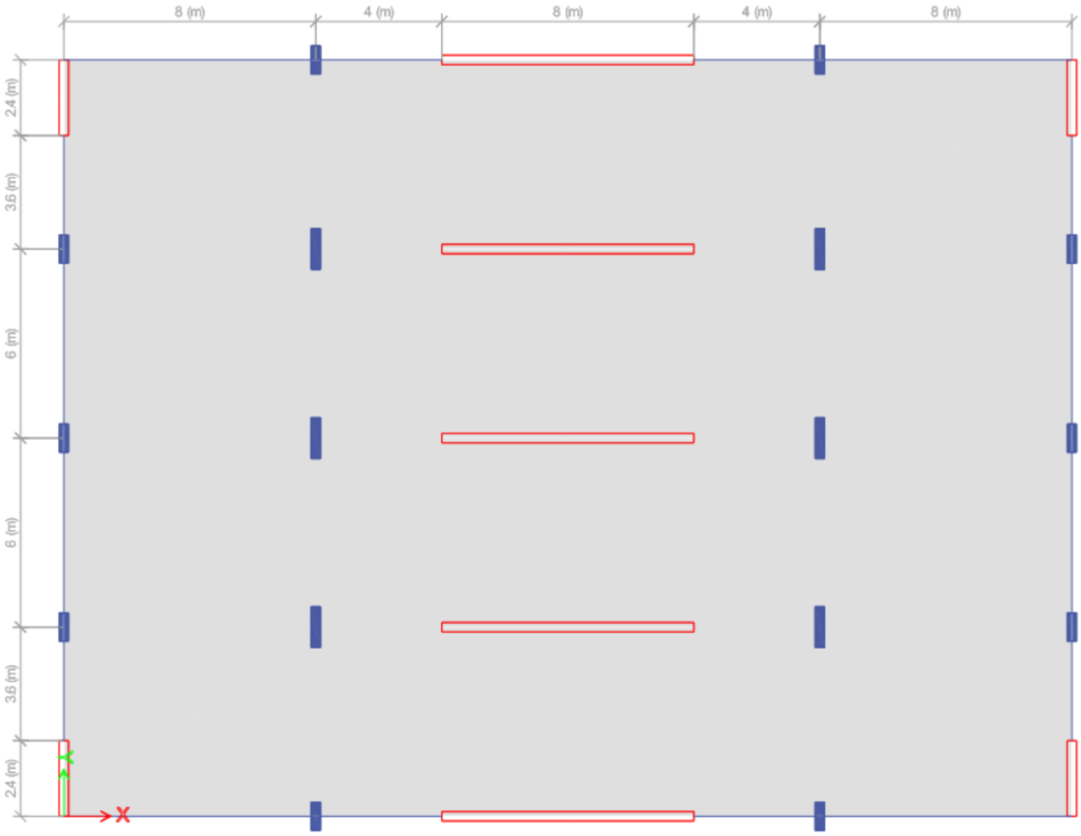


Figure 4 Shear Wall - Lateral Load System for Group 1

Group (2): Models of Group (2) include 10 Buildings with different number of stories (10,15,20) floors with both Moment-Resisting Frames at some floors as shown in Figure *5* and Shear Walls (at both X global direction and Y global direction) at other ones as shown in Figure 6.

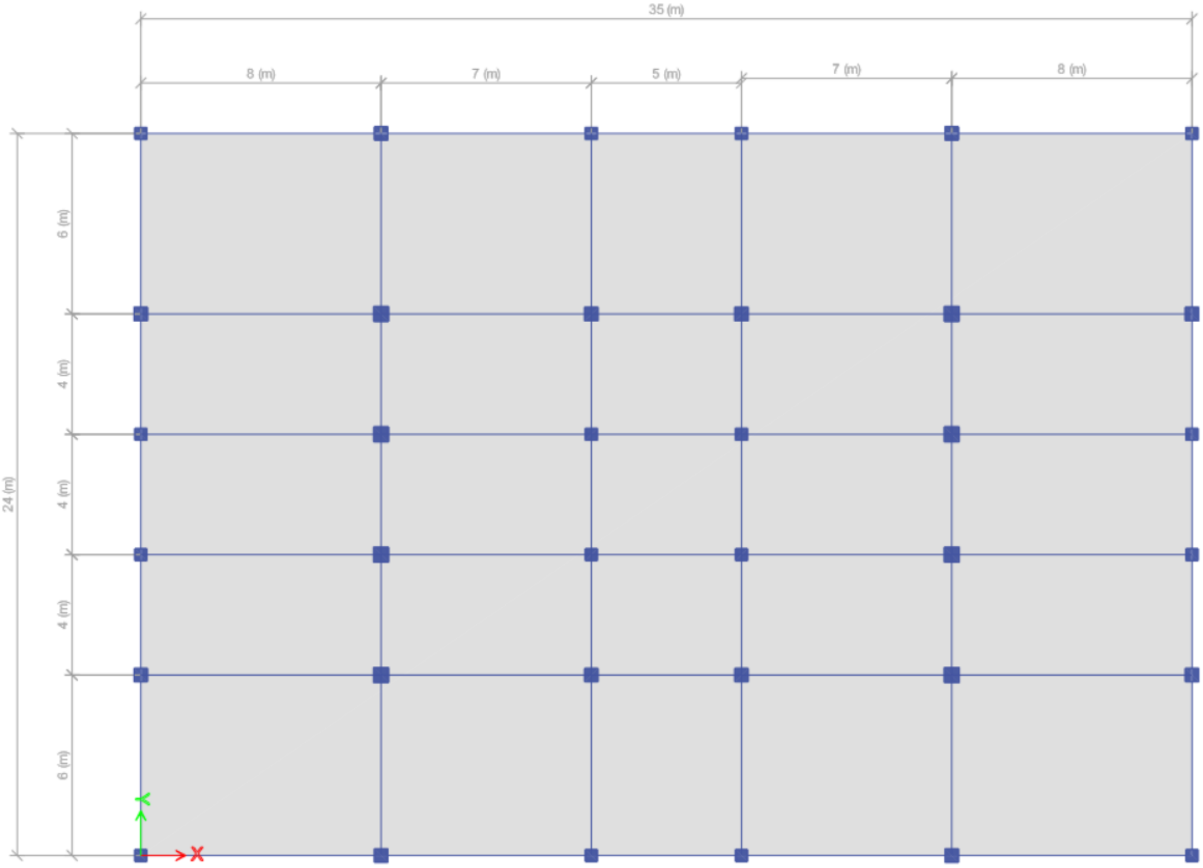


Figure 5 Moment Resisting Frame - Lateral Load System for Group 2

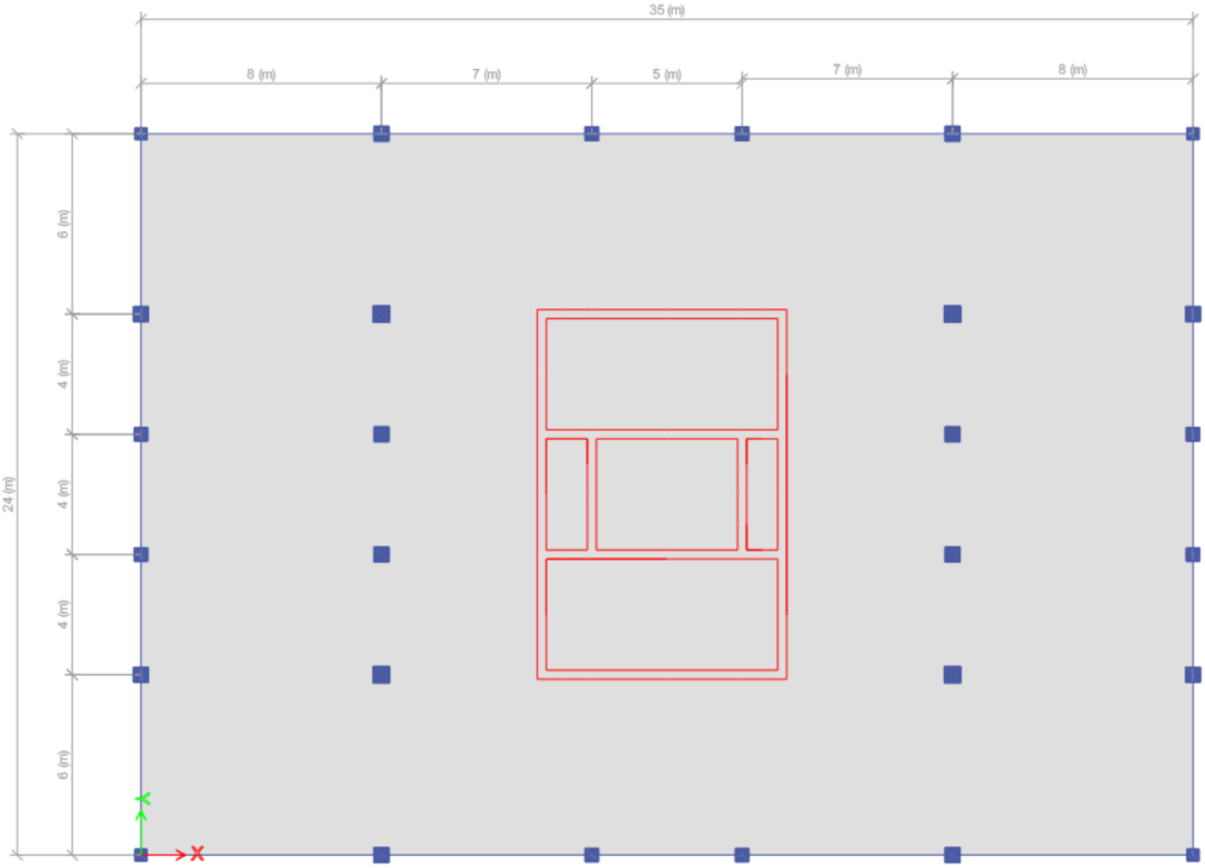


Figure 6 Shear Wall - Lateral Load System for Group 2

2.2.1 Material Properties

Concrete

- Concrete Cube Compressive Strength (F_{cu})= 30 MPa,
- Reinforced concrete density (Υ_c)= 25 kN/m^3,
- Modulus of Elasticity (E_c)= 24100 MPa,

Reinforcement: High-strength steel reinforcement is used.

- Longitudinal bars: F_y= 400 MPA, Fu= 600 MPa

2.2.2 Vertical Loads

All frames are designed as sway frames where loads are applied to nearby bays from both sides.

- 25cm thickness reinforced slab,
- Flooring Load = 0.2 ton/m2,
- Live load = 0.30 ton/m2,
- Density walls of 1.8 tons/m3 are used as line loads on beams = 1.2375 ton/m'

2.2.3 Seismic Loads

According to (ECP-201, 2012), seismic loads are defined as follows,

- Soil Type: C,
- Seismic Zone: 5B,
- Importance Factor: 1.0,
- Response Spectrum Type: Type 1,
- Ground Acceleration (ag): 0.3g

2.2.4 Dimensional Properties

The dimensions of the structure in this study were selected to ensure its safety under vertical loads, and the performance level was examined.

2.2.5 Modelling of the structure

The commercial finite element program (ETABS, 2021) was utilized, which carries out non-linear analysis and includes several non-linear static analysis methodologies stipulated by the codes. The process of creating ETABS finite elements models begins with the definition of the material properties for the frames, shells, and reinforcement. Next, the loads applied to the frames and shells, both vertical and lateral loads are presented. Next, the nonlinear pushover criteria are explained, along with the types and definitions of hinges that adhere to standards.

3. RESULTS AND DISCUSSION

3.1 Introduction

A variety of models of various heights and lateral load system configurations are analysed using the Pushover method. The purpose of this part is to compare these curves for various models to comprehend how the building responds and behaves variously. The static pushover curve provides us with base shear, displacement, and performance point results.

The models names that were selected for use in this study can be given in the form of G(number of group)(number of floors-the first lateral load resisting system- number of floors-the second lateral load resisting system).

For example, G1(5W-5F)
G1: represents group 1 as mentioned earlier.
5: number of floors for the shear wall system.
W: Shear Wall system.
5: number of floors for moment resisting system.
F: moment-resisting frame system.

3.2 Nonlinear Static Pushover Analysis

The non-linear static pushover analysis has been performed on the moment-resisting frames and shear walls using ETABS. For each distinct RC section in the case of columns, the P-M-M interaction diagram is used to analyze the impact of axial loads on plastic hinges. Typically, the pushover curve is utilized to display the results of the non-linear static analysis. The pushover curve is derived from the study frames' non-linear analysis and the bilinear idealization of each curve are shown from Figure 7 to Figure 16 for group 01, the same approach is used for group 02 graphs and full set of graphs could be retrieved from (Osama M, El kateb M, Zaher A,2024).

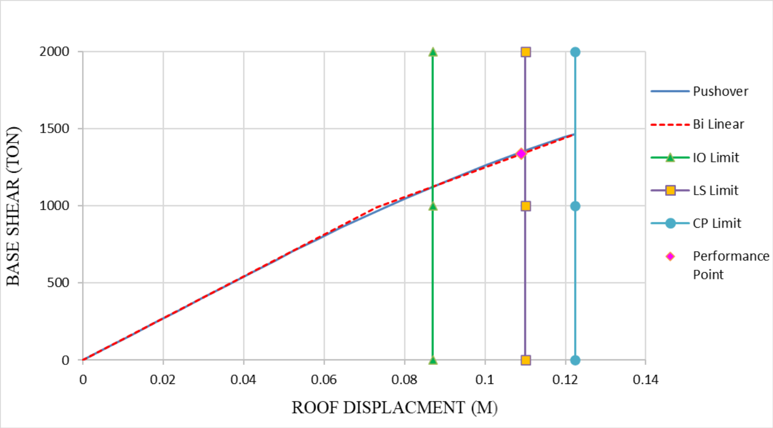


Figure 7 Pushover Curve for model G1 (5W-5F)

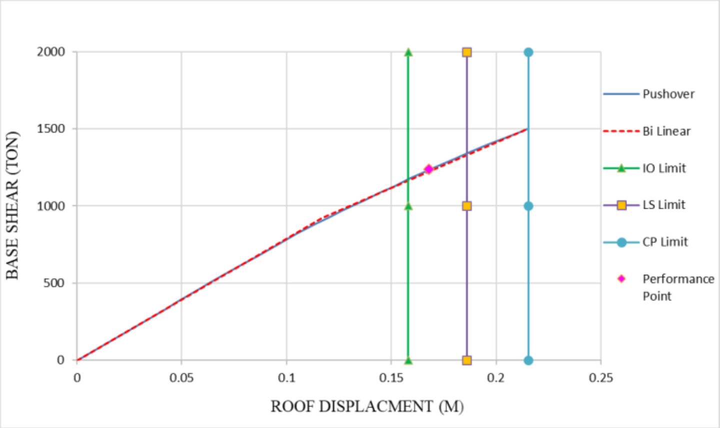


Figure 8 Pushover Curve for model G1 (5F-5W)

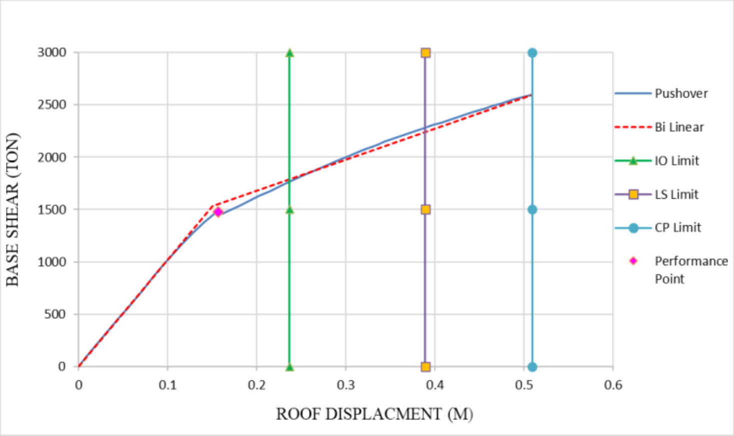


Figure 9 Pushover Curve for model G1 (5W-5F-5W)

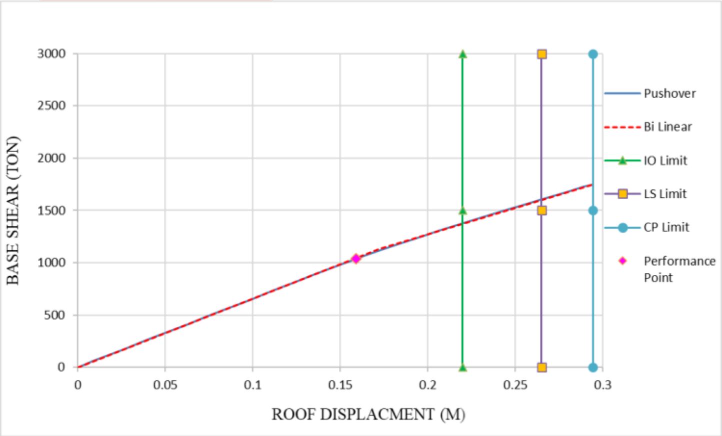


Figure 10 Pushover Curve for model G1(5F-5W-5F)

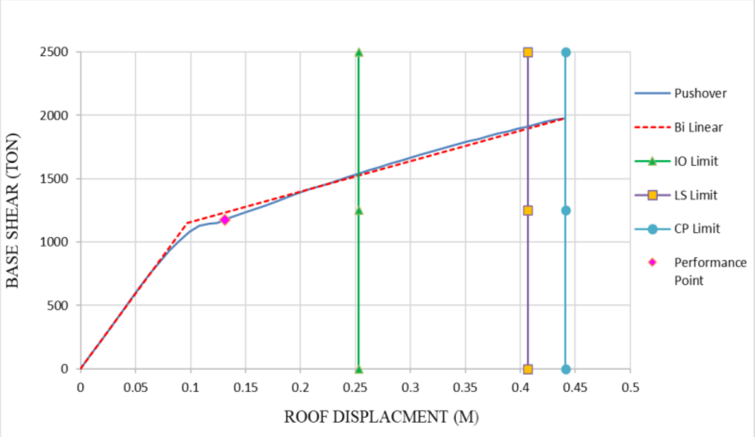


Figure 11 Pushover Curve for model G1 (10W-5F)

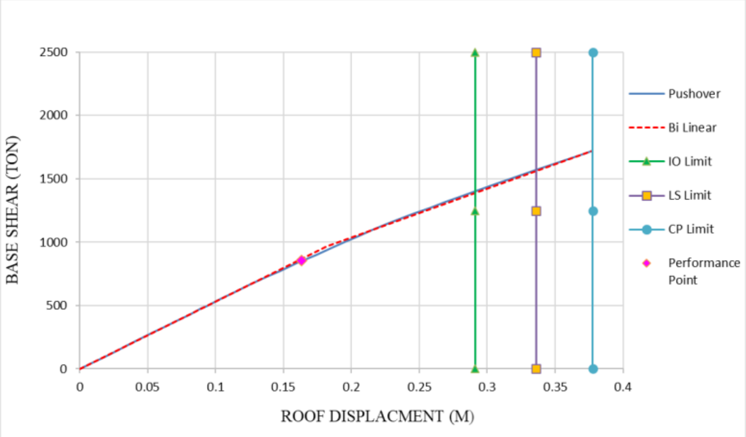


Figure 12 Pushover Curve for model G1 (10F-5W)

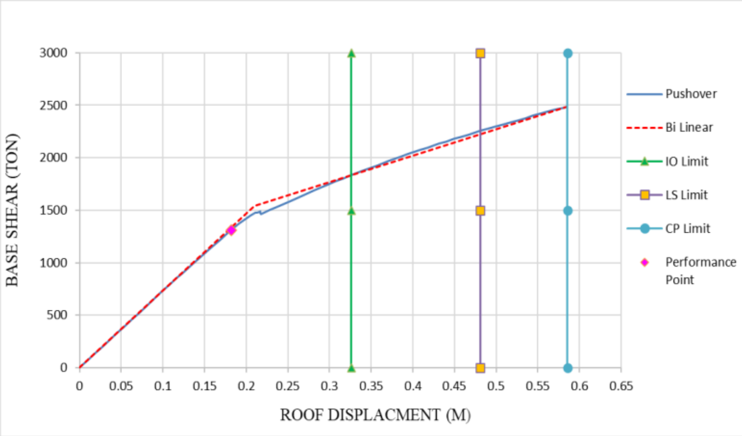


Figure 13 Pushover Curve for model G1 (5W-10F)

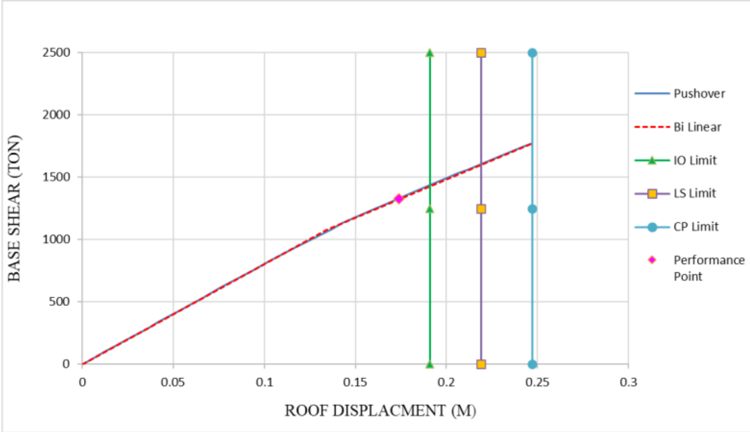


Figure 14 Pushover Curve for model G1 (5F-10W)

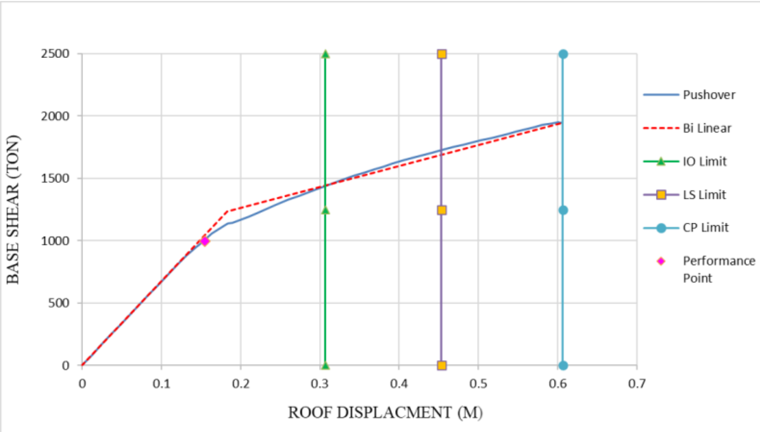


Figure 15 Pushover Curve for model G1 (10W-10F)

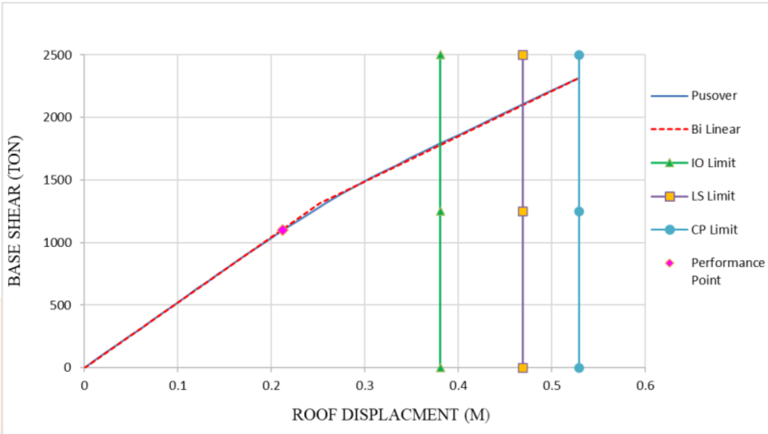


Figure 16 Pushover Curve for model G1 (10F-10W)

3.3 Determination of Performance and level of the structure

The capacity spectrum method can be utilized to determine the performance levels of buildings. The graphical comparison between the structural capacity and the seismic demand can be performed with the capacity spectrum method. The seismic demand is expressed by the response spectrum curve, while the pushover curve shows the lateral resisting capacity. The "Performance Point" is the point where the demand spectrum and the nonlinear pushover response intersect. To make sure that the damage to the structural and non-structural components is not greater than what is permitted, a performance check is required. The analyst may determine if the structure is safe or vulnerable and whether any strengthening needs to be done based on the position and condition of the performance point with the actual pushover curve. The demand and capacity curves are obtained via pushover analysis to determine the structure's performance point. The ADRS spectra (Acceleration -Displacement Response Spectrum) and the performance point of the buildings studied are shown from

Figure 17 to Figure 26 for group 01, the same approach is used for group 02 graphs and full set of graphs could be retrieved from (Osama M, El kateb M, Zaher A,2024).

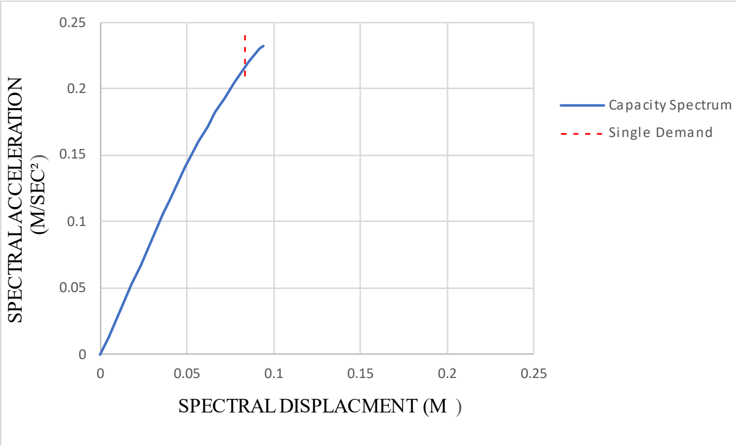


Figure 17 ADRS SPECTRA. For G1 (5W-5F)

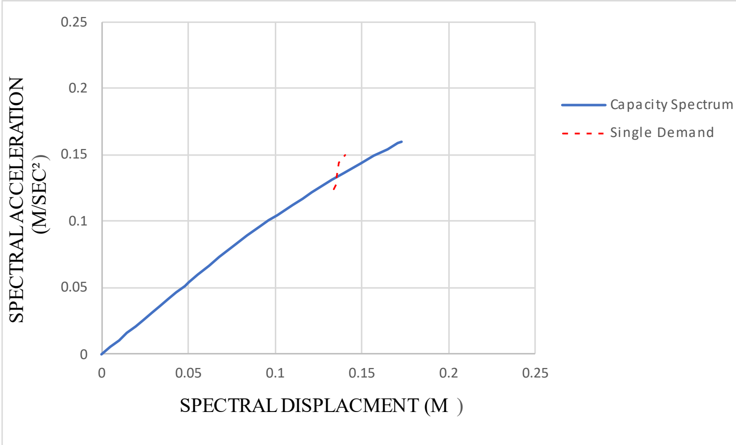


Figure 18 ADRS SPECTRA. For G1 (5F-5W)

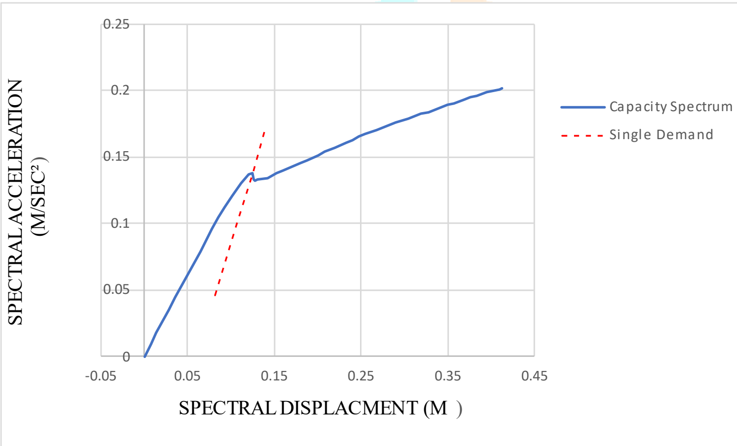


Figure 19 ADRS SPECTRA. For G1 (5W-5F-5W)

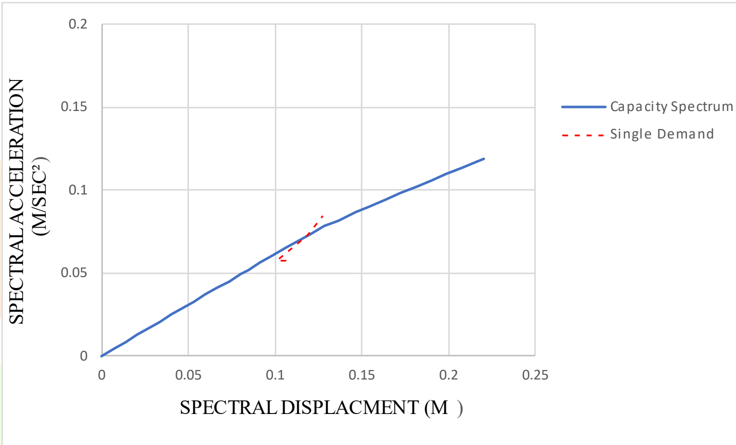


Figure 20 ADRS SPECTRA. For G1 (5F-5W-5F)

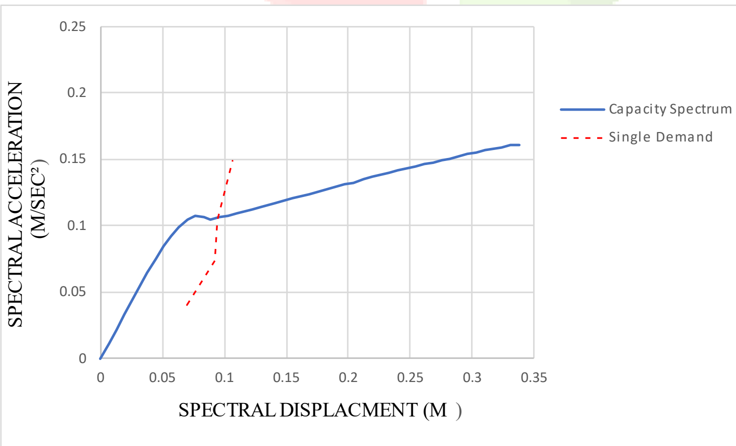


Figure 21 ADRS SPECTRA. For G1 (10W-5F)

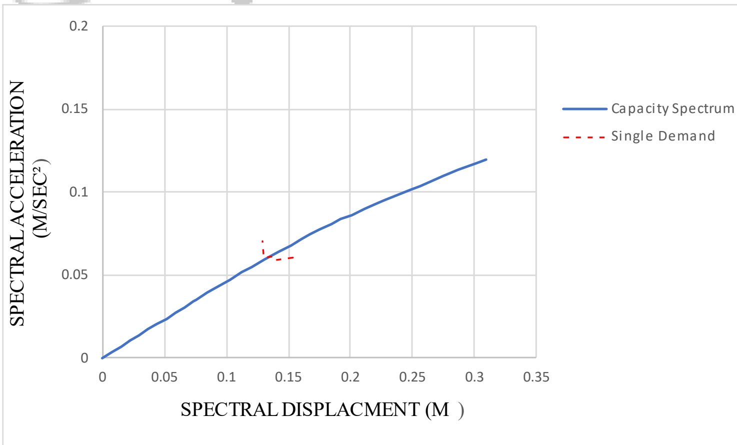


Figure 22 ADRS SPECTRA. For G1 (10F-5W)

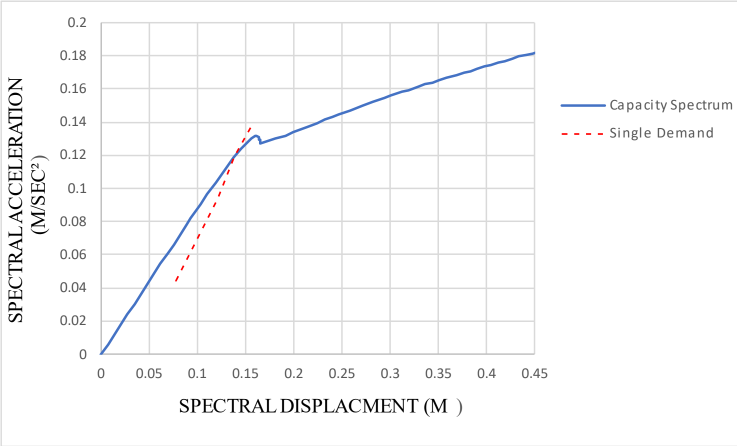


Figure 23 ADRS SPECTRA. For G1 (5W-10F)

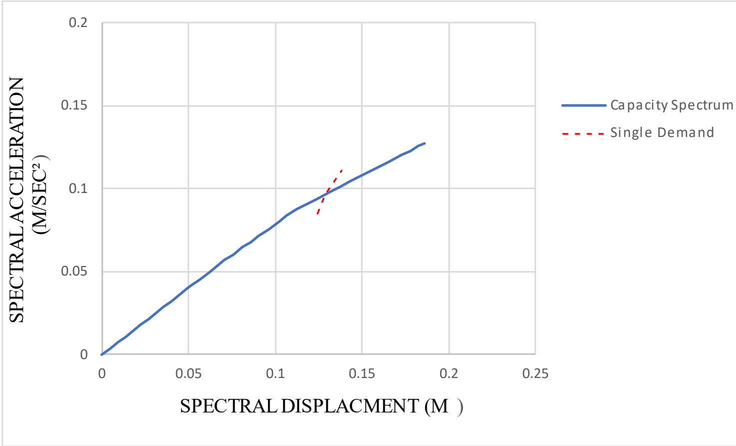


Figure 24 ADRS SPECTRA. For G1 (5F-10W)

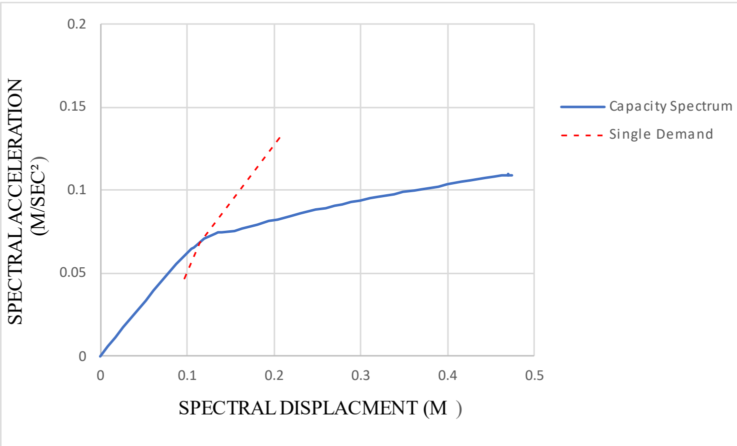


Figure 25 ADRS SPECTRA. For G1 (10W-10F)

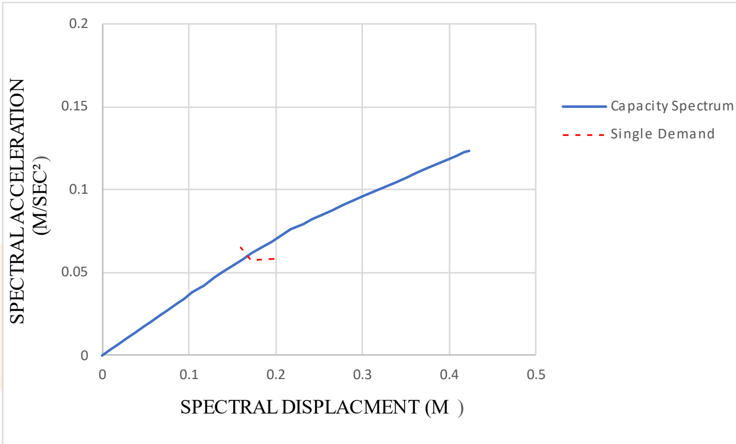


Figure 26 ADRS SPECTRA. For G1 (10F-10W)

According to the (ATC-40, 1996), to determine the degree of damage, the lateral displacement at the performance point must be compared to the displacement limitations as shown in Table 3 where Vi is the total calculated shear force in the story and Pi is the total gravity load (i.e. dead plus likely live load) at story i.

Drift Limit	Performance Level			
	Immediate Occupancy	Damage Control	Life Safety	Collapse Prevention (Structural Stability)
	1%	(1:2) %	2%	0.33 (Vi/Pi)

Table 3 Drift Limit According to (ATC-40, 1996)

3.4 Computation of Response Modification Factor

According to (ATC-19, 1996), the response reduction factor is expressed as a product of three factors:

$$R = R_S * R\mu * R_R \quad (1)$$

Where:

Overstrength factor (R_s): An indicator of the difference between materials, components, or structural system's actual and necessary strengths is called over-strength.

$$R_S = V_{max}/V_d \quad (2)$$

Ductility reduction factor (Rμ): It is a factor that lowers the elastic force to the structure's idealized yield strength level. The value of Rμ depends on the degree of ductility.

Short period	**T < 0.2 seconds**	**Rμ=1**
Moderate period	**0.2 < T < 0.5 sec**	**Rμ= √(2μ-1)**
Long period	**T > 0.5 seconds**	**Rμ=μ**

Redundancy factor (R_R): Multiple vertical lines of framing, each specifically built and detailed to convey earthquake-induced inertia pressures to the foundation, should make up a redundant seismic framing system. Using such systems, the relative (lateral) stiffness and strength characteristics of each frame determine how the lateral load is shared among them.

Lines of vertical seismic framing	**Draft redundancy factor**
2	0.76
3	0.86
4	1.00

Table 4 Redundancy factors (ATC-19, 1996)

This study aimed to determine the (R-Factor) for any lateral load-resisting system configuration that could be utilized to reflect the structure's real reaction in place of the response reduction factor required by seismic codes.

Model	Vmax (ton)	Vd (ton)	Δmax (m)	Δ_Y (m)	R_μ	$R_s=V_y/V_d$	R_R	R_ζ	R	Fundamental Natural Period (Sec)	Drift
G1 (5W-5F)	1466	989	0.122	0.073	1.67	1.48	1.00	1.00	2.48	2.465	0.0092
G1 (5F-5W)	1500	923	0.215	0.117	1.84	1.63	1.00	1.00	2.99	2.132	0.0114
G1 (5W-5F-5W)	2595	1446	0.509	0.150	3.39	1.79	1.00	1.00	6.09	3.314	0.0096
G1 (5F-5W-5F)	1749	1130	0.294	0.172	1.71	1.55	1.00	1.00	2.65	3.247	0.0075
G1 (10W-5F)	1976	1152	0.441	0.097	4.55	1.72	1.00	1.00	7.80	4.296	0.0053
G1 (10F-5W)	1723	970	0.378	0.182	2.08	1.78	1.00	1.00	3.69	3.313	0.0066
G1 (5W-10F)	2484	1539	0.585	0.210	2.79	1.61	1.00	1.00	4.50	3.415	0.0080
G1 (5F-10W)	1773	1080	0.247	0.134	1.84	1.64	1.00	1.00	3.03	3.215	0.0093
G1 (10W-10F)	1951	1233	0.601	0.183	3.28	1.58	1.00	1.00	5.20	5.235	0.0051
G1 (10F-10W)	2315	1316	0.529	0.252	2.10	1.76	1.00	1.00	3.69	3.878	0.0071
G2 (5W-5F)-X	2535	1146	0.16	0.094	1.70	2.21	1.00	1.00	3.77	1.516	0.0088
G2 (5W-5F)-Y	2565	1146	0.144	0.093	1.55	2.24	1.00	1.00	3.47		0.0087
G2 (5F-5W)-X	2436	1153	0.278	0.136	2.04	2.11	1.00	1.00	4.32	1.943	0.0099
G2 (5F-5W)-Y	2519	1153	0.234	0.108	2.17	2.18	1.00	1.00	4.73		0.0094
G2 (5W-5F-5W)-X	3751	1663	0.556	0.176	3.16	2.26	1.00	1.00	7.13	2.092	0.0091
G2 (5W-5F-5W)-Y	4050	1663	0.464	0.174	2.67	2.44	1.00	1.00	6.49		0.0093
G2 (5F-5W-5F)-X	3129	1819	0.438	0.219	2.00	1.72	1.00	1.00	3.44	2.671	0.0079
G2 (5F-5W-5F)-Y	3226	1819	0.364	0.174	2.09	1.77	1.00	1.00	3.71		0.0081
G2 (10W-5F)-X	3260	1657	0.594	0.135	4.40	1.97	1.00	1.00	8.66	1.692	0.0059
G2 (10W-5F)-Y	3758	1657	0.489	0.107	4.57	2.27	1.00	1.00	10.36		0.0070
G2 (10F-5W)-X	3565	1720	0.605	0.232	2.61	2.07	1.00	1.00	5.41	2.959	0.0051
G2 (10F-5W)-Y	3821	1720	0.537	0.186	2.89	2.22	1.00	1.00	6.41		0.0061
G2 (5W-10F)-X	4433	1826	0.658	0.252	2.61	2.43	1.00	1.00	6.34	2.348	0.0081
G2 (5W-10F)-Y	4782	1826	0.542	0.224	2.42	2.62	1.00	1.00	6.34		0.0073
G2 (5F-10W)-X	2949	1665	0.391	0.188	2.08	1.77	1.00	1.00	3.68	2.453	0.0093
G2 (5F-10W)-Y	3016	1665	0.291	0.142	2.05	1.81	1.00	1.00	3.71		0.0096
G2 (10W-10F)-X	3046	1875	0.771	0.166	4.64	1.62	1.00	1.00	7.55	2.653	0.0055
G2 (10W-10F)-Y	3312	1875	0.615	0.184	3.34	1.77	1.00	1.00	5.90		0.0053
G2 (10F-10W)-X	3937	1834	0.761	0.287	2.65	2.15	1.00	1.00	5.69	3.523	0.0064
G2 (10F-10W)-Y	4154	1834	0.621	0.224	2.77	2.26	1.00	1.00	6.28		0.0051

Table 5 Components for (R-Factor) for the studied models, and Fundamental Time period and drift

3.5 Conclusion

Based on the analysis results the following conclusions are summarized as follows:

1. As relative stiffness (between structure's stiffness in each global direction) increases, the response reduction factor increases.
2. Configuration of the lateral system in the vertical direction is sensitive and has a significant impact on the response reduction factor.
3. The lateral system becomes more ductile and effective in absorbing energy when it has sufficient stiffness in both directions.
4. When contemplating an earthquake-resistant system for a building of any height, it is preferable to use a moment-resisting floor system on the upper levels, which reduces drift.
5. The ductility and performance factors improve in tandem with the structure's height.
6. Real-life designs are likely to have a lower value of R than computed here due to irregular dimensions, poor quality control, and inability to follow ductile detailing guidelines.
7. An accurate assessment of the fundamental period (T) is required for calculating a realistic R of the structure.
8. The study only considers a single plan configuration in one seismic zone. The presented work uses deterministic parameters, but considerable statistical variances necessitate a reliability-based methodology.
9. The Egyptian design code should enhance the accuracy in the response reduction factor calculations. Regardless of the plan and vertical geometry, a single value of R for a specific frame type or shear wall type cannot be justified, also in many structural models, the response reduction factor is higher than the values presented in the Egyptian design code.

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