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Seismic Evaluation of an Existing Reinforced Concrete Building Based on FEMA P-2018 Methodology

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Abstract

The major earthquakes in Türkiye in recent years have shown the importance of assessing the building stock. However, modeling and assessing these structures realistically is a very time-consuming option. The FEMA P-2018 report, published by the Federal Emergency Management Agency (FEMA), provides a simplified methodology for evaluating older reinforced concrete structures that are likely to collapse, making building stock assessment efforts easier in terms of time. FEMA P-2018 procedures include the following steps: identifying the critical stories and components by estimating effective yield strength based on four collapse mechanisms, calculating drift demands and capacities of critical stories and components, evaluating collapse potential by comparing demand and capacities, determining column and wall ratings and final step is determining collapse potential rating of the structure. This study considers the behavior of an existing reinforced concrete building in Bayraklı, İzmir. The earthquake behavior of the structure was evaluated with FEMA P-2018 guidelines.

Keywords: *Reinforced concrete frames, Collapse mechanism, Building assessment*

1 INTRODUCTION

Major earthquakes have demonstrated the vulnerability of reinforced concrete (RC) buildings in regions with high seismic activity in Türkiye. A recent study [1] reported that the earthquakes showed that the primary causes of significant damage to reinforced concrete structures can be summarized as poor material and workmanship quality, poor detailing, lack of code compliance, soft story condition, etc.

Several studies [2, 3] have shown that the existence of older, seismically vulnerable concrete buildings, known as non-ductile reinforced concrete buildings, concerns public safety. Also, in FEMA P-2018 [3] it is stated that the 1971 San Fernando earthquake highlighted the poor performance of concrete frame buildings due to inadequate steel reinforcing in the design codes of that era. Structures need to be evaluated to reduce casualties and lower the cost of disasters. For this purpose, various seismic evaluation methods are used in practical applications in the literature. They can be classified into 3 tiers according to their evaluation detailing. FEMA P-154 [4] report includes the Tier 1 /Rapid Visual Screening (RVS) procedure that has been developed to identify, inventory, and screen potentially seismically hazardous buildings. The RVS procedure employs a method based on conducting a site survey of a building and completing a Data Collection Form based on visual observation of the building's exterior and, if it is accomplishable, its interior. The report [4] suggests that if they are identified as potentially hazardous, an experienced seismic design professional should perform a detailed evaluation of these structures to determine if they are seismically hazardous. A recent study by Erdil et al. (2019) [5] showed that Tier 2 (preliminary) assessment methods seem effective in identifying the seismic vulnerability of buildings, as they allow for an assessment of the seismic performance of an individual building when compared to Tier 1 assessment methods (street survey). In the Tier 3 evaluation approach, a complete analysis of the nonlinear response of the building to is performed, implicitly or explicitly, as defined in ASCE/SEI 41-17 [6].

A simplified approach, *Seismic Evaluation of Older Concrete Buildings for Collapse Potential*, was proposed by FEMA [3] to evaluate collapse risk by using simplified estimates of drift demands. This methodology aims to make the identification of the subset of buildings that are likely to have a high risk of collapse easier without the requirement for extensive testing or nonlinear analysis. The intention of this methodology is to identify the subsets of these buildings that are of lower relative risk of collapse, termed *lower seismic risk buildings*, or are of significantly higher relative risk of collapse, termed *exceptionally high seismic risk buildings*. This methodology focuses on structural deficiencies in vertical structural components. This approach assesses the reinforced concrete frame, bare frame, frame-wall, bearing wall, and infilled frame structures with heights less than 48.77 meters.

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2 SEISMIC EVALUATION METHOD

2.1 Summary of FEMA P-2018 Methodology

The methodology starts with classifying the structure as*: frame, frame-wall, bearing wall, or infilled frame-system* and determination of the *critical story.* Then, *horizontal spectral acceleration* for the recommended seismic hazard level at the site corresponding to the *effective fundamental period* is to be determined. After that, the *global demand-to-capacity ratio* and *global seismic drift demand* of the building, should be calculated. After getting these values, *component drift demand* and *component drift capacity* are to be determined. Finally, the *building rating* should be determined by taking the maximum *story rating* based on the average of the *component ratings*. The *Building Rating (BR),* as defined in FEMA P-2018, is utilized to assess and categorize the relative risk of buildings within an inventory, assigning them to one of three levels of relative risk: *exceptionally high seismic risk buildings* (if BR $\geq$ 0.7), *high seismic risk* buildings (if 0.7 $>$ BR $>$ 0.3), and *lower seismic risk buildings* (if 0.3 $\geq$ BR). A recommended flowchart for this methodology is shown in Figure 2.1.

2.2 Existing Reinforced Concrete Building

In this study, the seismic risk of an eight-story reinforced concrete building is investigated, which was built in 1992 in Bayraklı/İZMİR. The dimensions of the regular floor plan for the building, as shown in Figure 2.2, are 20.5 meters by 20.5 meters, and the height of the building is 23.1 meters. The ground-floor height is 3.5 meters, while the others are 2.8 meters. According to the structural drawings, the reinforced concrete structure was made of BS20 concrete and BCI steel type as per TS500/1984 [7]. However, field investigations showed that the compressive strength of concrete was 9 MPa. The building's site class was determined as “D”, based on ASCE 7-16 [8]. The building has infilled frames in its regular stories; however, the ground floor has only 5 infilled frames (Figure 2.3). Thus, the system is classified as an “infilled frame system”. For plastic mechanism analysis, loads and component strengths need to be calculated. First, the expected gravity load is calculated using the equation taken from FEMA P-2018 [3]. The equation is presented below (1). With this equation, all expected gravity loads on columns and concrete walls were calculated.

$$P_G = P_D + 0.25P_L \tag{1}$$

Component flexural and shear strengths are calculated based on expected material properties. Concrete and reinforcing steel properties used in component strength calculations shall be expected material properties, as the methodology requires; the expected compressive strength of concrete and expected yield strength of reinforcement are calculated with the following equations (2, 3), and the results are 9.5 MPa and 275 MPa, respectively.

$$f_{ce}^{'} = 1.5 f_c^{'} \tag{2}$$

$$f_{ye}^{'} = 1.25 f_y^{'} \tag{3}$$

Then, component strengths are calculated, which include columns and walls flexural strength considering P-M interaction, columns and walls shear strength (Eqs. 4, 5), beams flexural and shear strength, columns capacity-limited shear strength, masonry infilled panel strength, and infilled bay strength (Eqs. 6, 7).

$$V_n = k\left(\frac{A_v f_{ye}^{'} d}{s} + \lambda\left(\frac{0.5\sqrt{f_{ce}^{'}}}{l_{\text{inf}}/d}\sqrt{1+\frac{P_g}{0.5A_g\sqrt{f_{ce}^{'}}}}\right)0.8A_g\right) \tag{4}$$

$$V_n = (\alpha_c \lambda \sqrt{f_c^{'}} + \rho_t f_y) A_{cv} \tag{5}$$

$$V_m = P_{\text{inf}}^{grav}\mu + A_{w,\text{inf}}C \tag{6}$$

$$V_{\text{inf},solid} = \max\left(V_m, P_{\text{inf}}^{\max}\mu + V_{lc}\right) \tag{7}$$

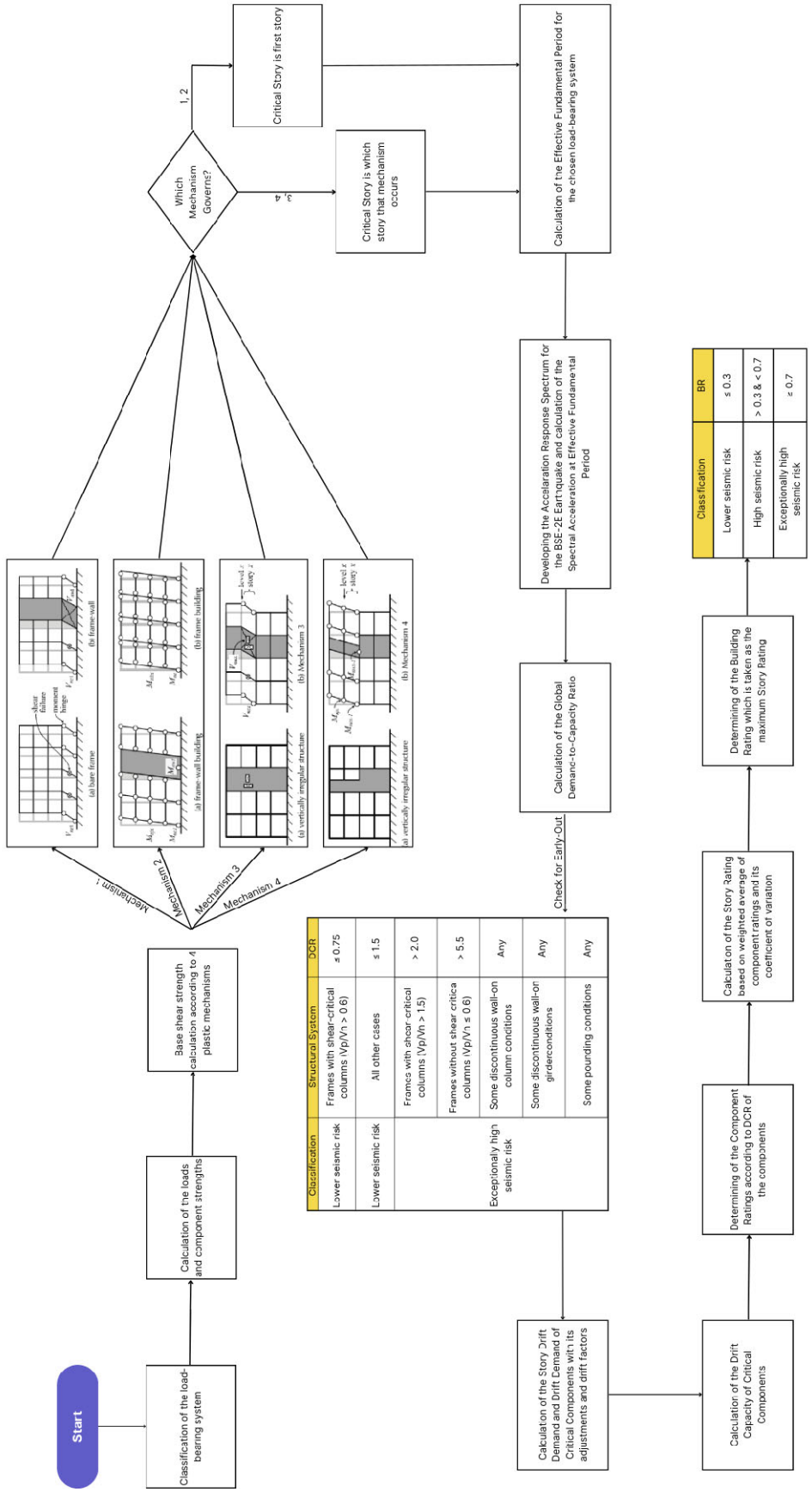


Figure 2.1. Flowchart for the FEMA P-2018 Methodology

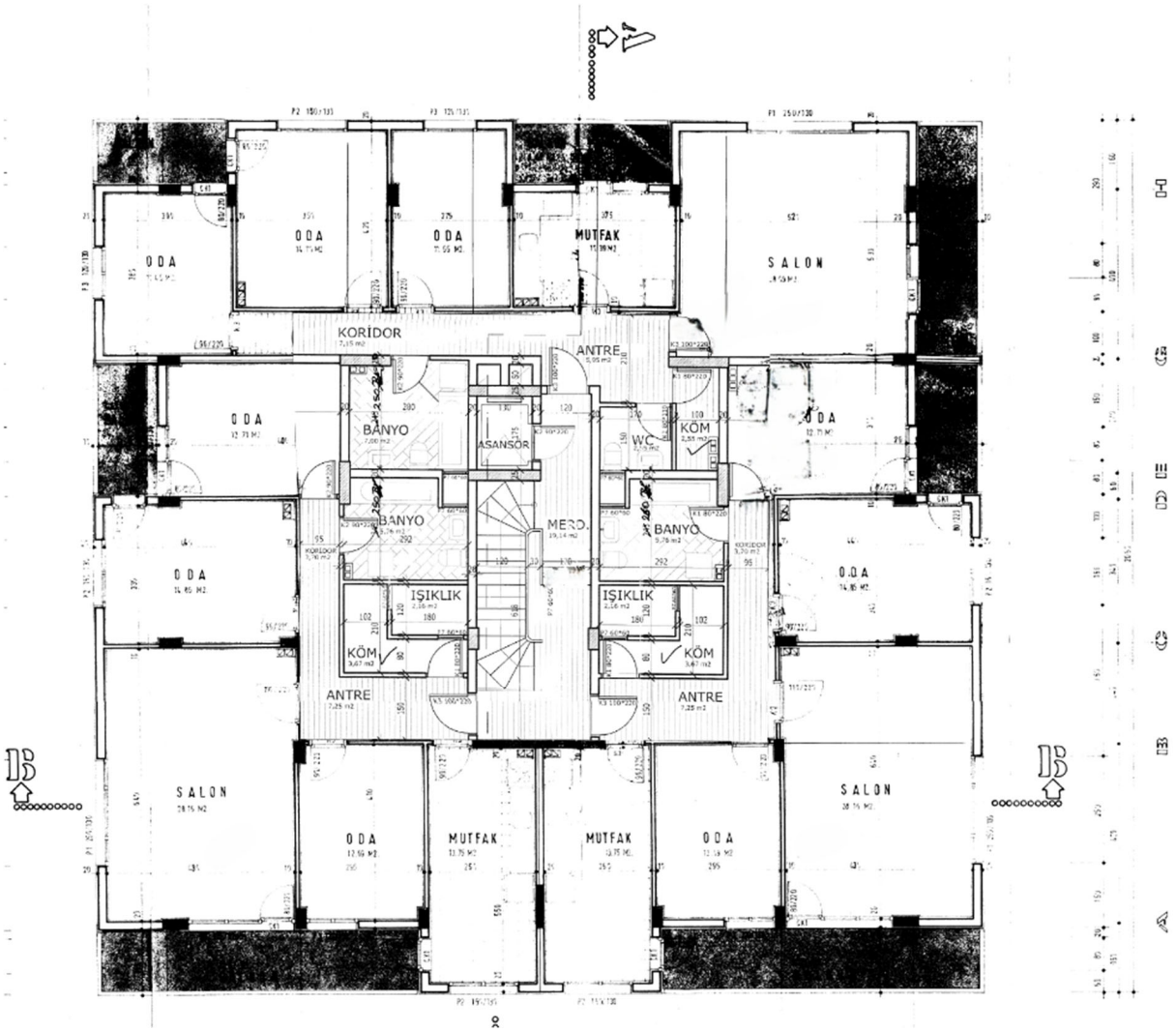


Figure 2.2. Architectural floor plan (2-8 stories)

3 RESULTS

The governing mechanism is determined after obtaining the values below. Plastic mechanism base shear strengths for Mechanism 1 and Mechanism 2 are shown in Table 3.1. Additionally, the plastic mechanism base shear strength for the infilled frame system is also stated in Table 3.1. Thus, Mechanism 2 governs in the X direction, and Mechanism 1 governs in the Y direction. Although the base shear strength for Mechanism 2 is lower than Mechanism 1, it is greater than three-quarters (3/4) of the base shear strength for Mechanism 1. Therefore, Mechanism 1 should be taken as the controlling mechanism in the Y direction according to FEMA P-2018 [3]. Also, the plastic mechanism base shear strength for the infilled frame is calculated. When all these shear strengths are compared, plastic mechanism base shear strength for the X direction equals 4151.4 kN, and for the Y direction, it equals 4556.6 kN.

Table 3.1. Base shear strengths

Cases	X	Y
Mechanism 1	6961.14 kN	5270.98 kN
Mechanism 2	4151.4 kN	4765.52 kN
Infilled Frame	5083.96 kN	4556.6 kN

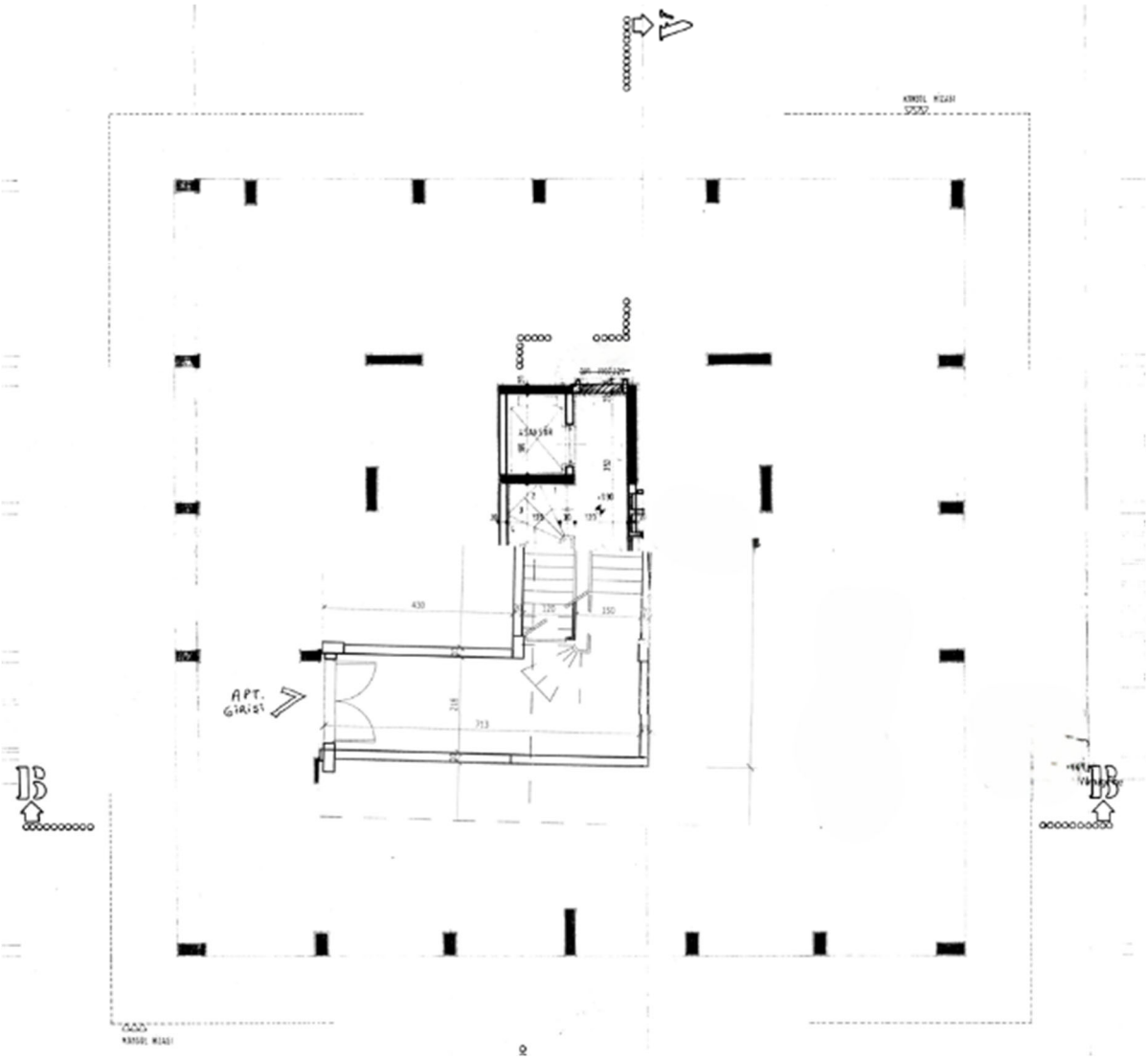


Figure 2.3. Architectural plan of the ground floor

The effective fundamental period is calculated using the formulas (Eqs. 8- 11) for frame systems, frame-wall systems, and infilled frame systems, which are taken from FEMA P-2018 [3]. The effective fundamental period is 0.482 s and 0.937 s for the X and Y directions, respectively.

$$T_e = 0.07\left(h_n\right)^{0.5}\left(\frac{V_y}{W}\right)^{-0.5} \tag{8}$$

$$T_e = 0.0026\frac{h_n}{\sqrt{C_w}} \le T_e \tag{9}$$

$$C_w = \frac{100}{A_{base}}\sum_{i=1}^{j}\frac{A_{wi}}{1+0.83\left(\frac{h_{wi}}{l_{wi}}\right)^2} \tag{10}$$

$$T_e = \left(1.6 - 0.6\frac{V_{p1}}{V_{p1,bare}}\right)T_{e,bare} \tag{11}$$

The recommended seismic hazard level for evaluation is the ASCE/SEI 41-17 Basic Safety Earthquake BSE-2E, which corresponds to a 5% probability of exceedance in a 50-year period [3]. In Türkiye there is no such seismic hazard level. There is a 2% probability of exceedance in a 50-year period and a 10% probability of exceedance. A study [9] has shown the relation between two peak ground acceleration levels and two different corresponding periods. That relation was used with modification to get a 5% probability, as shown below (12, 13). Thus, short-period spectral acceleration and spectral acceleration corresponding to a 1-sec period are calculated. Design spectral accelerations are determined by using the appropriate coefficients for the soil classification. With these values, an elastic design spectrum is obtained. Spectral values are given in Table 3.2, and the spectra are presented in Figure 3.4. Spectral acceleration values at effective fundamental periods for both orthogonal directions are determined. Spectral accelerations are 1.194 g and 1.124 g, respectively, for the X and Y directions.

$$\frac{S_{a,R1}}{S_{a,R2}} = \left(\frac{T_{R1}}{T_{R2}}\right)^k \tag{12}$$

$$S_{a,N} = \left(\frac{T_N}{T_{R2}}\right)^k \times S_{a,R2} \tag{13}$$

Table 3.2. Spectral values

Return Period	S_s	S_1	S_{DS}	S_{D1}
2475	2.120	0.543	1.696	1.148
975	1.472	0.368	1.194	1.053
475	1.111	0.273	1.123	0.801

The global demand-to-capacity ratio was calculated using spectral accelerations, effective yield strengths, building total seismic weight, and effective mass factor, which equals 1.0 for the 8-story infill wall system. The ratio is 8.4 and 7.2 for the X and Y directions. For early identification, the capacity-limited (plastic) shear strength to nominal shear strength ratio was calculated for all vertical components at the critical story (the first story). The ratio is lower than 0.6 for both orthogonal directions. Also, the global demand-to-capacity ratio is greater than 5.5 for both orthogonal directions. Therefore, this building could be identified as an “exceptionally high seismic risk building”.

If the current structure is to be ranked, seismic demand and the capacity of the components need to be calculated. Global seismic demand needs to be calculated first to calculate component seismic demand. It can be calculated using the global demand-to-capacity ratio, effective fundamental period, spectral acceleration, and soil coefficient. Using these calculations, global seismic drift demand is calculated as 0.137 meters and 0.274 meters for the X and Y directions. Then, story drift demand is calculated with α coefficient. Coefficient α modifies story drifts considering the number of stories in a building, the yield mechanism, and whether the story is critical. Story drift demand is 0.109 meters and 0.219 meters, respectively, for the X and Y directions. Lastly, P-Δ adjustment needs to be calculated. With this adjustment, the adjusted story drift demand is 0.14 meters and 0.367 meters, respectively, for the X and Y directions.

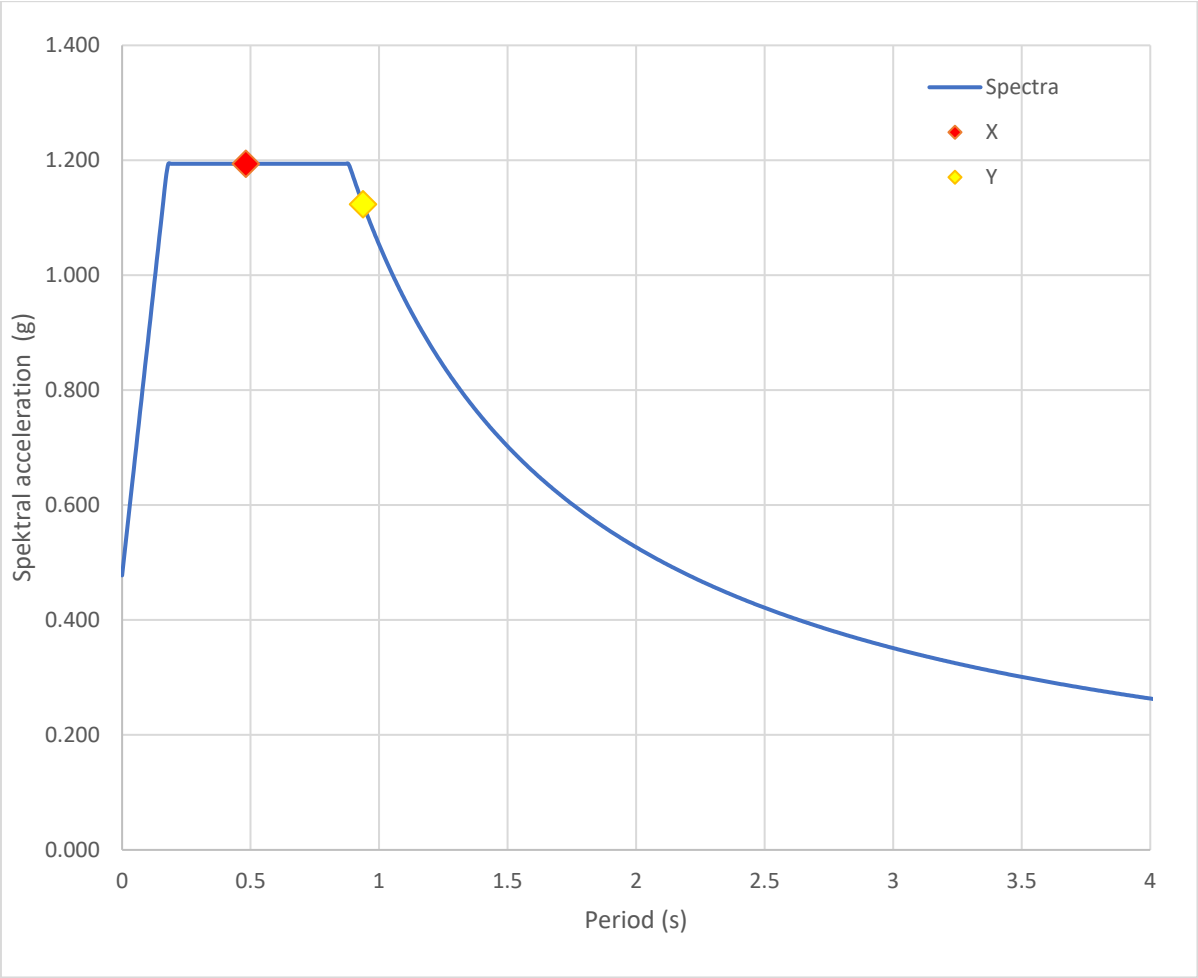


Figure 3.4. Elastic Response Spectra

Component drift demand is calculated using the drift factor. Column drift factors are given in Table 3.3, and the vertical wall drift factor is taken as 1.0. Component drift capacity is calculated using the equation below (14). Using the component drift demand-to-component drift capacity ratio, components can be rated according to Table 3.4.

Table 3.3. Column drift factors

Ratio of Column Strengths to Beam Strengths	Column Drift Factor
≤ 0.6	0.85
1	0.70
≥ 2.4	0.30

$$\Delta_c = l_u\left(\theta_c + 0.01\right) \tag{14}$$

Using the component ratings and fraction of gravity loads supported by that component, the weighted average story rating can be calculated. Then, adjustment based on the coefficient of variation (standard deviation of all the vertical component ratings at a story divided by the weighted mean vertical component rating at a story) is done. Lastly, story rating is calculated using the adjusted average of column and wall ratings in the story. Story rating is calculated as 0.67 and 0.9 for the X and Y directions, respectively. The building rating, BR, is taken as the maximum story rating. Therefore, BR is 0.9, which corresponds to “exceptionally high seismic risk building”.

Table 3.4. Column and wall rating

Drift Demand to Drift Capacity Ratio	Rating
$\Delta_D / \Delta_C \leq 0.25$	0.0
$0.4 \geq \Delta_D / \Delta_C > 0.25$	0.1
$0.5 \geq \Delta_D / \Delta_C > 0.4$	0.2
$0.7 \geq \Delta_D / \Delta_C > 0.5$	0.3
$0.9 \geq \Delta_D / \Delta_C > 0.7$	0.4
$1.1 \geq \Delta_D / \Delta_C > 0.9$	0.5
$1.4 \geq \Delta_D / \Delta_C > 1.1$	0.6
$1.8 \geq \Delta_D / \Delta_C > 1.4$	0.7
$2.5 \geq \Delta_D / \Delta_C > 1.8$	0.8
$3.0 \geq \Delta_D / \Delta_C > 2.5$	0.9
$\Delta_D / \Delta_C > 3.0$	0.93

4 CONCLUSION

In this study, an 8-story reinforced concrete building with infilled frames built in 1990's is evaluated using the FEMA P-2018 evaluation methodology. *Base effective yield strength* and *critical story* of the building based on plastic mechanism analysis and procedures are determined for 3 possible mechanisms. When all these shear strengths are compared, *Mechanism 2* is the governing plastic mechanism since base shear strength for the X direction is lower than the strength values in other cases. Also, the global demand-to-capacity ratios (8.4 and 7.2) are greater than 5.5 for both orthogonal directions. It indicates that the building is classified as an *exceptionally high seismic risk building*. Besides, the *building rating (BR)* is calculated as 0.9, which is the highest possible rating for collapse. Therefore, exceptionally high seismic risk buildings should be given the highest priority for more detailed evaluation in an inventory.

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