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Quantifying the response modification factor of RC buildings: A comparative study using nonlinear pushover analysis

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Abstract

Earthquake-resistant design relies on controlled inelastic structural behavior, typically incorporated into seismic codes through the response modification factor (R), which reduces elastic force demands by accounting for ductility, overstrength, and energy dissipation. However, extensive research indicates that the assumed ductility levels in code-based R values are often not representative of older reinforced concrete (RC) buildings lacking proper seismic detailing, particularly in high-risk regions such as Istanbul. This study evaluates the applicability of conventional R-factor assumptions for an existing six-story RC residential building constructed in the 1980s in Istanbul's Kadıköy district. Nonlinear static pushover analysis was employed to assess the building's lateral load capacity and inelastic performance under two contrasting scenarios: a code-based high ductility assumption (R = 8) and a reduced, more conservative value (R = 3).

Results demonstrate that the higher R value leads to seismic demand estimates slightly below the structural capacity, falsely indicating adequate performance and safety, which contradicts observed earthquake damage patterns in the studied structure. In contrast, the lower R value produces demand levels significantly exceeding the building's capacity, aligning more closely with real damage observations and highlighting deficiencies in ductility and torsional resistance due to structural irregularities. These findings are consistent with prior studies emphasizing the sensitivity of R to detailing quality and structural configuration. The study underscores that adopting high R values for substandard existing buildings can result in unconservative and potentially unsafe design assessments. It is therefore recommended that reduced R values—on the order of 3 or lower—be adopted for existing RC structures in high seismic zones to better reflect their actual inelastic capacity and ensure more reliable performance evaluation.

Keywords: Response modification factor (R); Nonlinear static pushover analysis; Seismic performance; Ductility; Torsional resistance

1. Introduction

Earthquake-resistant design fundamentally relies on controlled inelastic behavior rather than a purely elastic response, as designing for full elasticity under strong ground motion would result in impractically large structural members. Consequently, modern seismic codes, such as ASCE 7-22 [2], utilize the response modification factor (R) to reduce elastic seismic demands to realistic design forces by accounting for ductility, overstrength, and energy dissipation. In reinforced concrete (RC) frame systems, the R factor directly governs design base shear and member sizing; however, the assumption of high ductility is heavily dependent on detailing quality. Many existing RC buildings lack essential

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features—such as proper confinement and "strong-column–weak-beam" behavior—leading to an actual inelastic capacity that is significantly lower than code assumptions [10, 17]. While research by Mondal et al. [15] and Sharifi et al. [18] has demonstrated that geometry and detailing significantly influence R values, much of the existing literature [4, 9, 14, 16, 20] focuses on code-compliant structures. This creates a safety gap for older building stocks in high-hazard regions like Istanbul, where post-earthquake observations consistently reveal an overestimation of energy dissipation capacity [5, 6, 7, 12, 13]. To address this, the present study investigates a 1980s-era six-story RC building from Istanbul's Kadıköy district, using nonlinear static pushover analysis to evaluate seismic performance under contrasting R-factor assumptions. By comparing a conventional high R-factor with a more conservative lower value, this research aims to determine which better reflects the observed behavior of vulnerable existing structures.

2. Structural Model Description

To examine the influence of the response modification factor (R) on seismic performance, an existing six-story reinforced concrete (RC) frame building was analytically modeled to represent typical residential construction in Istanbul's seismically active Kadıköy district. The selected structure reflects common late-twentieth-century construction practices, characterized by simple geometry and conventional moment-resisting frame behavior (Figure 1). The building was idealized as a 3D RC frame system with a total story height of 3.0 m and a clear height of 2.5 m, consistent with regional residential standards of that era [1].

To ensure a realistic representation, material properties—including concrete compressive strength and reinforcing steel yield strength—were adopted from in-situ experimental data reported by Aras et al. [1]. These properties were derived from core samples and laboratory testing of a severely earthquake-damaged building currently undergoing demolition, thereby capturing the actual mechanical characteristics of the existing building stock.

The analytical model was developed using ETABS (Computers and Structures, Inc.), where beams and columns were represented as frame elements. To accurately simulate inelastic behavior under seismic loading, nonlinear hinge properties were assigned at critical regions, such as beam-column joints, in accordance with established guidelines for lumped plasticity modeling [4, 8]. Gravity loads were incorporated to reflect realistic service conditions, including a superimposed dead load of 2 kN/m^2, partition wall loads (5 kN/m for exterior and 3 kN/m for interior walls), and a live load of 2 kN/m^2. These loads were applied in combination with lateral loads during the nonlinear analysis to capture the effects and the interaction between vertical and seismic demands.

For the concrete quality estimation of the building, 18 cores were taken from its columns. The test results gave the average compressive strength as 15.2 MPa with a standard deviation of 3.2 MPa. Reinforcement details were also searched, and it was seen that S220 steel grade (characteristic yielding stress of 220 MPa) was used in the structural elements. Observed structural deficiencies, including inadequate lap splice lengths and insufficient confinement, were compounded by severe reinforcement corrosion. These factors must be accounted for by applying appropriate degradation factors to the material properties.

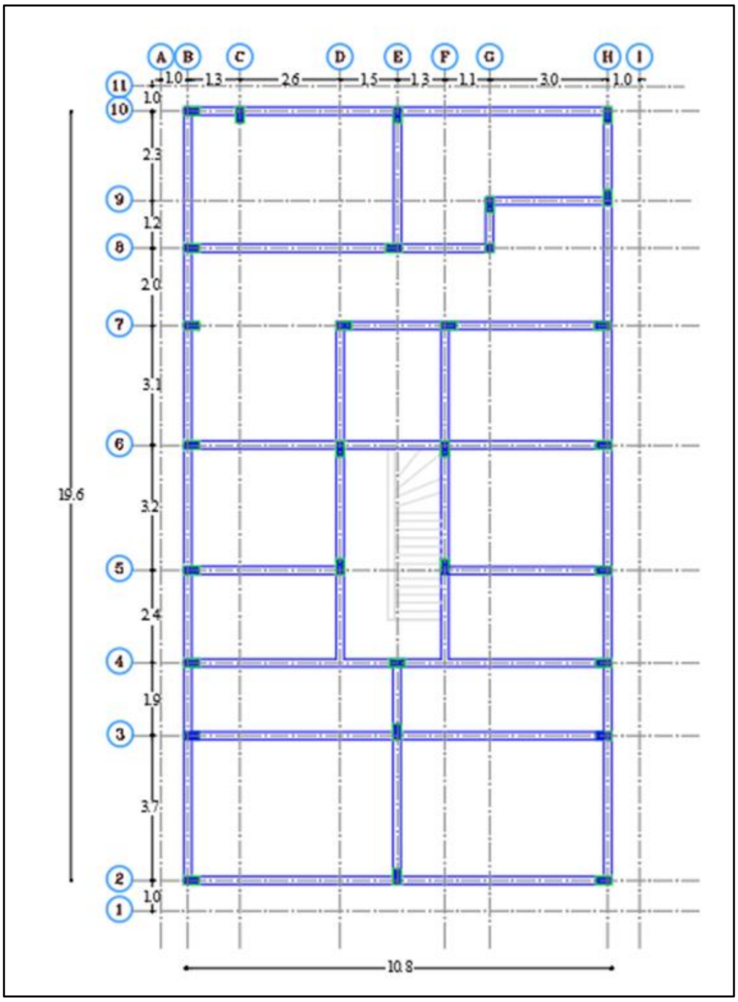

Figure 1 Typical Floor Layout Showing Beam and Column Configuration (in meters)

3. Seismic Hazard and Response Spectrum

The seismic demand in this study is defined by site-specific response spectra representing the hazard conditions of Istanbul's Kadıköy district. This region is a high-risk seismic zone due to its proximity to the Marmara segment of the North Anatolian Fault, one of the world's most active fault systems. Given the historical occurrence of destructive earthquakes and the ongoing threat to the Istanbul metropolitan area, an accurate representation of seismic demand is essential for a realistic structural performance assessment.

As illustrated in Figure 2, the response spectrum characterizes the expected ground motion at the site, plotting spectral acceleration (S_a) as a function of the structural period (T), typically assuming 5% damping. These spectra represent the maximum expected response of structures under earthquake excitation and serve as the foundation for calculating the elastic design base shear [2, 20]. The selected spectrum incorporates regional seismicity, fault proximity (near-source effects), and local soil conditions to provide a reliable representation of demand for the studied RC frame.

In accordance with modern seismic design procedures, structural inelastic behavior is accounted for via the response modification factor (R). This factor reduces the elastic seismic forces derived from the response spectrum to practical design-level forces. This reduction fundamentally assumes that the structure possesses sufficient ductility and over strength to dissipate energy through controlled damage without reaching a state of collapse.

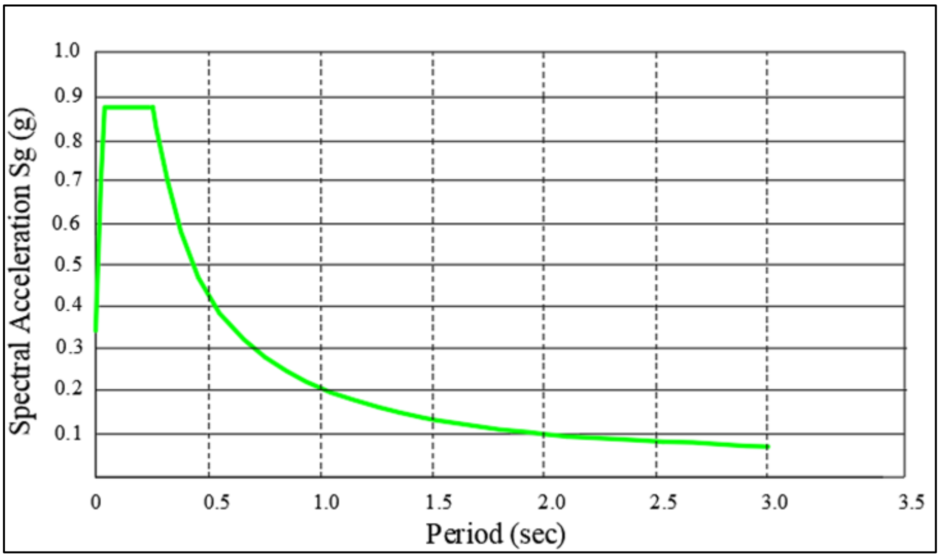

Figure 2 Response Spectrum of the Building Site

4. Nonlinear Static Pushover Analysis

This study evaluates nonlinear structural response through displacement-controlled pushover analysis, utilizing lumped plasticity hinges at member ends to model damage in accordance with ASCE 41-13 guidelines. These hinges characterize nonlinear behavior via a five-point backbone curve—comprising the origin (A), yield (B), ultimate capacity (C), residual strength (D), and total failure (E)—while joint performance is classified into Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) levels to assess seismic safety, as detailed in Section 5.2.

To analyze the building's response, the multi-curved capacity results are simplified into an equivalent bilinear curve through a formal idealization process. This procedure ensures area equivalence to represent consistent energy dissipation, utilizing an effective stiffness line drawn through 60% of the yield strength. The intersection of the elastic and post-yield segments defines the system's yield point, while the secondary slope accounts for either strain hardening or softening.

5. Results and Discussion

5.1. The Pushover Curve

The pushover curve illustrated in Figure 3 represents the nonlinear static response of the building under increasing lateral loads. This curve defines the critical relationship between base shear and roof displacement, providing essential insights into the structure's strength, stiffness, ductility, and overall seismic performance.

Initially, the linear segment of the curve reflects a purely elastic response with a high initial stiffness (K_i) of 48003 kN/m. In this stage, the structure remains undamaged. As roof displacement reaches the 38 mm value, the curve deviates from linearity, marking the yield point where plastic hinges begin to form. This transition signifies the entry into the inelastic range, where the building initiates-controlled damage and energy dissipation.

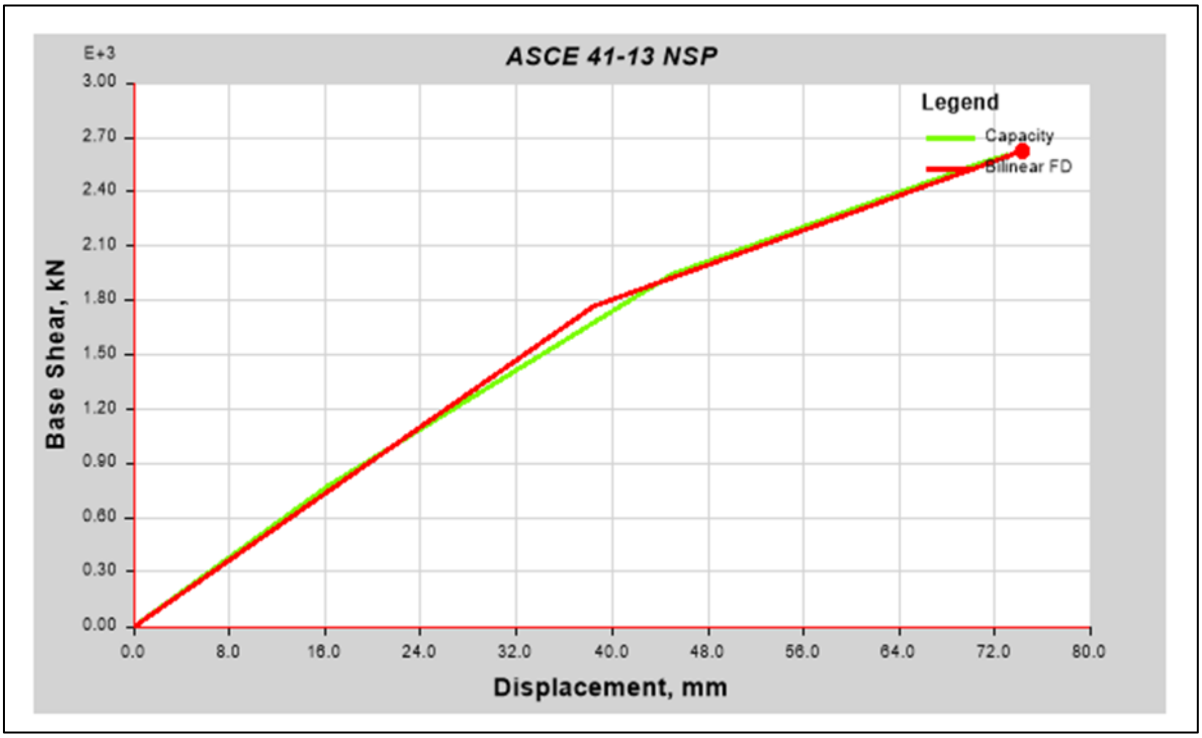

Figure 3 Pushover Curve ASCE 41-13 Displacement Modification

Past the yield point, a reduced slope marks a stiffness loss. The resulting effective stiffness (K_e) is 45998 kN/m. Internal force redistribution allows the structure to resist further loads. The capacity point occurs at a displacement of approximately 76 mm. At this limit, the recorded base shear reaches 2621 kN.

This point defines the maximum seismic demand the system can sustain. Exceeding these specific values would compromise overall safety limits. The data confirms structural performance under peak design stress. To facilitate assessment, a bilinear idealization simplifies this behavior.

The force displacement Green Curve in Figure 3 captures the actual nonlinear response of a structure through smooth, high-accuracy simulations of real-world behavior. The Red Curve simplifies this into a piecewise linear model, using ASCE 41 idealization rules to facilitate practical design and code calculations. Together, they bridge the gap between complex structural reality and the approximate, standardized data required for engineering analysis.

5.2. Hinge Performance Levels

To evaluate the structural state of the building under increasing seismic demand, the performance levels are defined according to the ASCE/SEI 41-13 [3] color-coding scheme illustrated in Figure 4. The spatial distribution of plastic hinges developed during the nonlinear static pushover analysis is further detailed in Figures 5 through 10, providing a visual mapping of structural vulnerability.

The performance criteria categorized by the hinge response are defined as follows:

Grey (IO — Immediate Occupancy): This level represents the onset of nonlinearity with minimal structural damage. The system retains nearly all of its original strength and stiffness, ensuring the building remains safe for continued occupancy immediately following the seismic event.

Green (LS — Life Safety): This state indicates that the structure has sustained significant damage, including permanent deformation of members. However, a critical margin against global collapse is maintained. While residents may sustain injuries due to falling hazards or structural degradation, the risk of fatalities remains low.

Red (CP — Collapse Prevention): This critical threshold signifies that the structure is on the verge of total failure. Although the building remains standing, it possesses very little residual strength or stiffness. At this stage, the gravity-load-resisting system is severely compromised, and any additional lateral loading or a major aftershock could trigger a progressive collapse.

By monitoring the transition of hinges through these stages, the analysis identifies whether the building maintains the "Strong-Column–Weak-Beam" requirements or if localized column failures dominate the response.

Figure 4 Plastic Hinge Performance Level Color-Coding

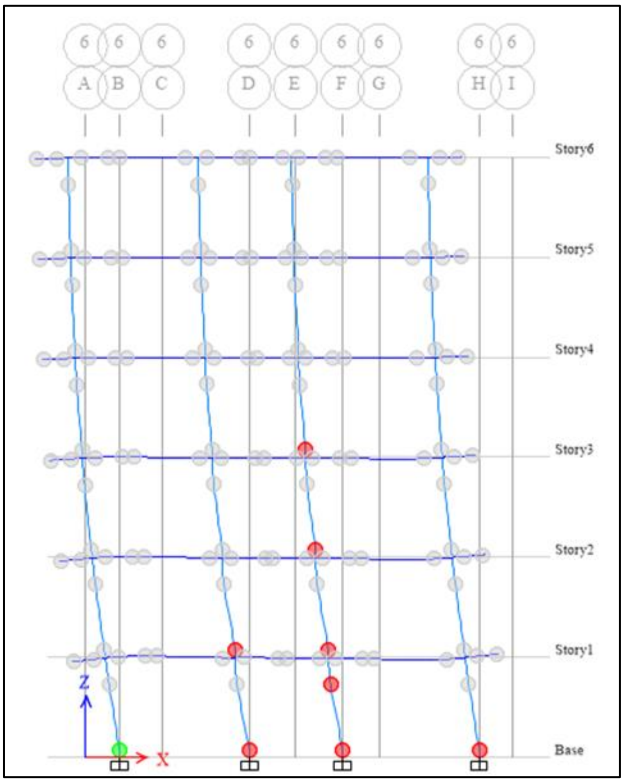

Figure 5 Plastic Hinge Formation and Distribution at Grid 6

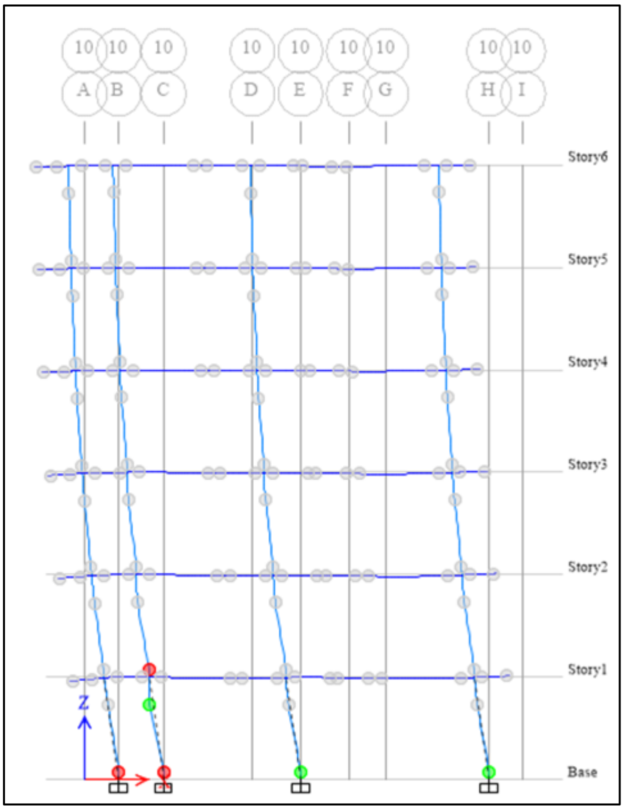

Figure 6 Plastic Hinge Formation and Distribution at Grid 10

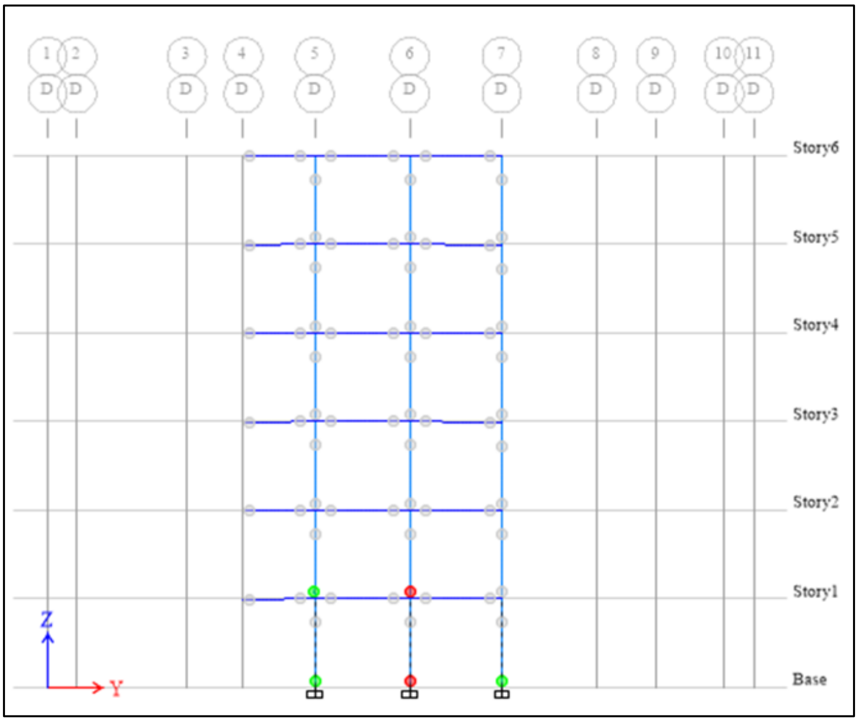

Figure 7 Plastic Hinge Formation and Distribution at Grid D

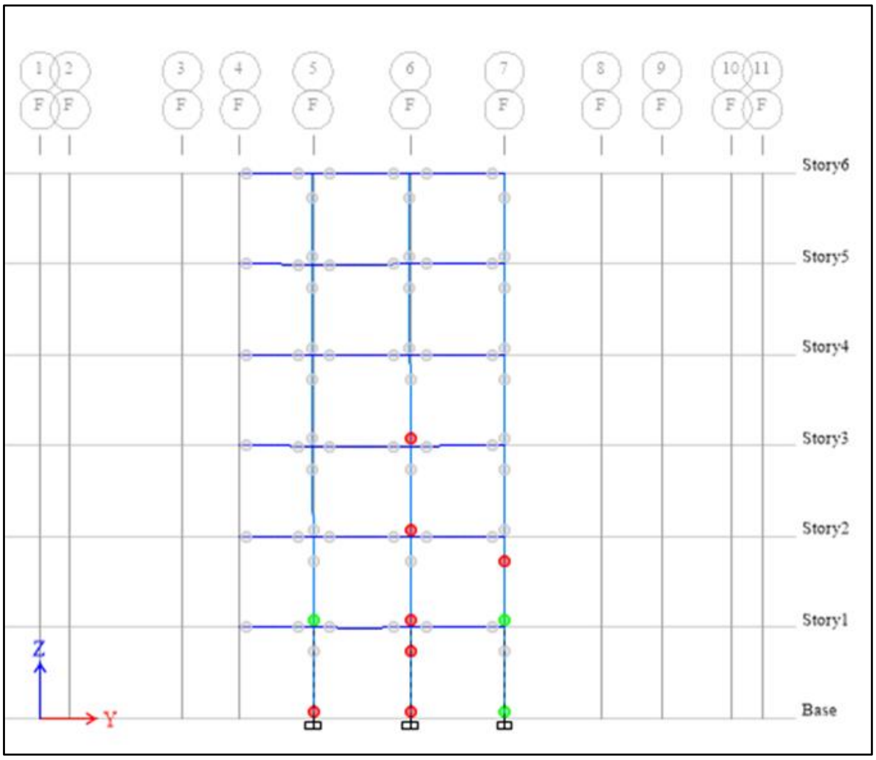

Figure 8 Plastic Hinge Formation and Distribution at Grid F

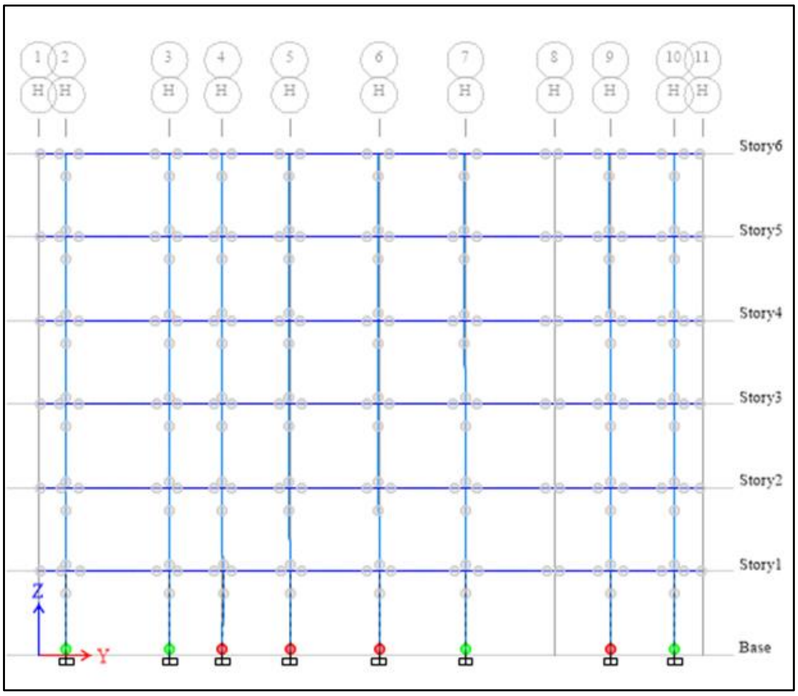

Figure 9 Plastic Hinge Formation and Distribution at Grid H

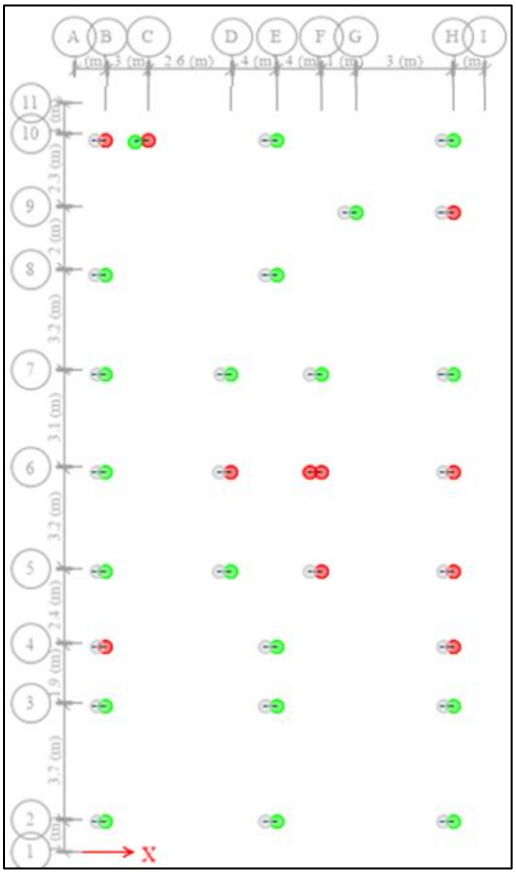

Figure 10 Plastic Hinge Formation and Distribution at Ground Floor

Many of the Red (CP) and Green (LS) hinges are located directly on the vertical elements (columns) at the ground floor level. When columns reach Collapse Prevention (CP) at the ground floor while the beams are still relatively intact, it creates a "story mechanism." This means the entire ground floor could laterally displace and collapse under the weight of the floors above, which remain stiff.

Because the red (CP) and Green (LS) hinges are appearing in columns rather than beams at the ground level, this indicates a potential "Strong Beam-Weak Column" mechanism. This is generally undesirable in seismic design as it can lead to a soft-story collapse.

To calculate the Ductility Ratio (μ), we look at the relationship between how far the building can deform before failure versus when it first starts to damage (yield). Using the values from your ASCE 41-13 interpretation, we can define the formula as:

$$\mu = \frac{\Delta_u}{\Delta_y} = \frac{76}{38} = 2.0$$

Modern seismic codes for ductile reinforced concrete structures, such as Special Moment Frames, typically target a displacement ductility ratio (R) between 3.0 and 8.0. In contrast, the calculated ratio of 2.0 for this building is notably low, characterizing a 'limited ductility' response where the structure reaches its ultimate capacity shortly after the initial formation of plastic hinges.

Because most seismic codes rely on high Response Modification Factors (R) to justify reduced design forces, a significant safety gap emerges when the design assumes while the actual ductility is only 2.0. This discrepancy is the primary reason for the observed structural failure: the building simply lacked the deformation capacity required to meet the inelastic demands implied by its design classification.

5.3. Demand vs. Capacity Analysis

The comparison between the building's seismic demand and structural capacity reveals a deceptive safety margin. While the ultimate lateral capacity (V= 2621 kN) exceeds the standard code-based demand (R=8) of 1895 kN with a safety ratio of 1.38 , the structure fails catastrophically when evaluated against a more realistic low-ductility demand (R=3) of 5054 kN.

This significant capacity deficit is compounded by a "Strong-Beam–Weak-Column" mechanism. At a 76 mm displacement demand, the base columns are forced into a Collapse Prevention (CP) state, preventing the ductile energy dissipation typically achieved through preferred beam hinging. Data from Table 1 further confirms that torsional effects intensify this risk, leading to localized failures in the foundation and vertical elements. Ultimately, the structure lacks the necessary ductility to safely transition into nonlinear behavior under high seismic demands. These findings suggest that current code-based -factor assumptions significantly overestimate the building's actual energy dissipation capacity and overall structural integrity in high-hazard scenarios.

Such building currently lacks the ductility required for its design -factor of 8, functioning instead at an R-value of 2. Without intervention, a major earthquake would likely cause a total collapse. To improve seismic performance, it is recommended to jacket the columns—using a maximum stirrup spacing of 10 cm—to upgrade the building's ductility to R=3. Strengthening these low-ductility structures to achieve an R-factor of 3 or less is essential to prevent catastrophic column failure.

Table 1 Base Reactions—Equivalent Static Seismic (EQX) and Pushover Analyses

Output Case	**Case Type**	**FX**	**FY**	**FZ**	**MX**	**MY**	**MZ**
		kN	kN	kN	kN-m	kN-m	kN-m
EQX (R=8	LinStatic	1895	0.00	0.00	0.00	25534	22145
EQX (R=3)	LinStatic	5054	0.00	0.00	0.00	68092	53722
PUSHOVER	NonStatic	2621	0.00	12695	150346	81405	30991

6. Conclusions

This study evaluated the impact of the response modification factor (R) on the seismic performance assessment of a typical 1980s-era reinforced concrete residential building in Istanbul. Based on the analytical results and observed field performance, the following conclusions are drawn:

Nonlinear static pushover analysis effectively captured the lateral load capacity and inelastic behavior of the building's reinforced concrete frame.

Applying a high response modification factor (R=8) resulted in seismic demand levels slightly below the calculated structural capacity. This suggested the building would remain safe, a finding that contradicted the actual observed damage, indicating that the assumed ductility was significantly overestimated.

When the factor was reduced to R=3, the calculated base shear and torsion demand far exceeded the pushover capacity. This result aligned more accurately with the severe structural damage observed in reality.

The discrepancy between the torsional demand at R=3 and the available torsional capacity, as evidenced in Table 1, primarily stems from the building's structural asymmetry.

The findings highlight that adopting high factors for existing structures with limited ductile detailing can lead to dangerously unconservative demand estimates.

While the building's lateral load capacity aligns with a design response modification factor of R=8, the actual available ductility is only sufficient for R=2. This discrepancy poses a significant risk of total collapse during a major seismic event.

To mitigate this, the structural ductility must be enhanced to meet at least an R=3 standard. This can be achieved through column jacketing with a transverse reinforcement (stirrup) spacing of 10 cm or less. This intervention will increase both the lateral capacity and ductility, prevent brittle column failure mechanisms and ensure the building meets the requirements for $R \leq 3$.

Compliance with ethical standards

Disclosure of conflict of interest

No conflict of interest to be disclosed.

References

[1] Aras, F., Akbaş, T., Çeribaşi, S., &Çatbaş, F. N. (2021). Modelling of infill walls in reinforced concrete buildings under low-amplitude vibrations. Proceedings of the Institution of Civil Engineers - Structures and Buildings, 176(7), 487–499. https://doi.org/10.1680/jstbu.20.00310

[2] American Society of Civil Engineers (ASCE). (2022). Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7-22). Reston, VA: ASCE.

[3] American Society of Civil Engineers (ASCE). (2014). Seismic evaluation and retrofit of existing buildings. ASCE/SEI 41-13, Reston, VA.

[4] Applied Technology Council (ATC). (1995). Structural Response Modification Factors. ATC-19 Report, Redwood City, CA: Applied Technology Council.

[5] Boore, David M., & Atkinson, Gail M. (2008). Ground-motion prediction equations for the average horizontal component of PGA, PGV, and 5%-damped PSA at spectral periods between 0.01 s and 10.0 s. Earthquake Spectra, 24(1), 99–138.

[6] Crowley, H., Colombi, M., Crempien, J., Pinho, R., (2011). Application of a simplified prioritization scheme to earthquake risk assessment in urban areas. 14th World Conference on Earthquake Engineering.

[7] Erberik, M. A., (2008). Fragility-based assessment of typical RC buildings in Turkey. Engineering Structures, 30(5), 1360–1374.

[8] Erdik, M. (2013). Earthquake risk in Istanbul. Natural Hazards, 68(2), 1083–1107. https://doi.org/10.1007/s11069-013-0676-z

[9] Federal Emergency Management Agency (FEMA). (2005). Improvement of Nonlinear Static Seismic Analysis Procedures (FEMA 440 / ATC-55). Washington, D.C.

[10] Federal Emergency Management Agency (FEMA). (2000). Pre-standard and Commentary for the Seismic Rehabilitation of Buildings. FEMA 356. Washington, DC.

[11] Fardis, M., N., (2009). Seismic Design, Assessment and Retrofitting of Concrete Buildings: Based on EN-Eurocode 8. Dordrecht: Springer Science & Business Media.

[12] Karakale, V., & Layas, F. M. (2025)., Rationale for updating earthquake design forces in modern building codes. World Journal of Advanced Engineering Technology and Sciences, 14(1), 103–109. https://doi.org/10.30574/wjaets.2025.14.1.0005

[13] Layas, F. M., &Karakale, V., (2025). Systemic seismic vulnerabilities: Key construction failures and mitigation strategies. World Journal of Advanced Engineering Technology and Sciences, 17(3), 121–128. https://doi.org/10.30574/wjaets.2025.17.3.1542

[14] Miranda, E., & Bertero, V.,(1994). Evaluation of strength reduction factors for earthquake-resistant design. Earthquake Spectra, 10(2), 357–379.

[15] Mondal, A.; Jain, S. K.; Murty, C. V. R., (2013). Evaluation of response reduction factor for RC moment-resisting frames. Engineering Structures, 52, 195–208.

[16] Newmark, N. M. & Hall, W. J., (1982). Earthquake Spectra and Design. Berkeley, CA: Earthquake Engineering Research Institute.

[17] Paulay, T., & Priestley, M. J. N., (1992). Seismic Design of Reinforced Concrete and Masonry Buildings. New York, NY: John Wiley & Sons.

[18] Sharifi, S., Toopchi-Nezhad, H., (2018). Seismic response modification factor of RC-frame structures based on limit state design. International Journal of Civil Engineering, 16(10), 1391–1402.

[19] Soyman, M., (2018). Evaluation of Seismic Response Modification Factors for Reinforced Concrete Buildings. PhD Thesis, Middle East Technical University, Ankara, Turkey.

[20] Uang, C., (1991). Establishing R factors for building seismic provisions. Journal of Structural Engin