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# Comprehensive seismic evaluation of existing buildings using ASCE 41-17 standards

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***Abstract***
*This study addresses the lack of comprehensive seismic evaluations for eight-story reinforced concrete buildings in high-seismic zones, such as Jakarta, using the ASCE 41-17 standard. The research evaluates the seismic performance of a 35-year-old office building through a tiered analysis approach, including Tier 1, Tier 2, and Tier 3. The study aims to identify structural deficiencies and propose retrofitting measures to meet modern seismic standards. Defects in soft story behavior and overturning stability were among the five and fourteen items in the Tier 1 assessment that showed noncompliance. In Tier 2, linear analysis revealed critical ductility demands, with Demand-to-Capacity Ratios (DCR) exceeding permissible limits in most structural elements. The nonlinear pushover analysis conducted in Tier 3 revealed an insufficient structural capacity to withstand high seismic loads. Maximum inter-story drifts in the X and Y directions were 2.321% and 2.319%, respectively, surpassing Life Safety standards. The findings indicate that the building's seismic performance falls between the Life Safety and Collapse Prevention levels, emphasizing the urgent need for retrofitting to enhance its resilience. This research presents a comprehensive framework for integrating global standards and local seismic conditions to enhance the safety and performance of existing structures in high-risk areas.*



***Keywords:***
*ASCE 41-17;*
*Retrofitting;*
*Seismic Performance;*
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## INTRODUCTION

Despite Indonesia's high seismic activity, no research explicitly addresses the comprehensive seismic evaluation of the country's existing eight-story reinforced concrete buildings using ASCE 41-17 standards. This study is the first to complete a Tiers 1–3 assessment, providing a detailed understanding of structural deficiencies and necessary retrofitting strategies. Previous research has generally concentrated on specialized materials, such as cold-formed steel structures [1] or industrial steel buildings [2], leaving reinforced concrete structures in high-seismic areas unexplored. These studies focus on the relationships between material qualities, structural systems, and regional seismic hazards. However, the lack of a complete assessment covering Tiers 1–3 may result in critical structural vulnerabilities being neglected. However, the lack of a complete evaluation covering Tiers 1–3 may result in essential structural vulnerabilities being neglected.

Additionally, research based on standards such as AISC 360-10 or local regulations like SNI 1726:2019 [3] often lacks broader applicability, as it does not fully address performance-based design for medium-rise buildings with moment-resisting frames [4]. This gap highlights the need for a seismic assessment integrating global standards with local seismic conditions. This study comprehensively assesses reinforced concrete structures in Jakarta, incorporating local seismic conditions as per ASCE 41-17, to recommend practical retrofitting techniques that enhance seismic resistance.

Prior research involving assessing the five-story reinforced concrete structure XYZ employed SNI 1726:2002 for seismic-resistant design. However, with the adoption of SNI 1726:2019, changes in seismic maps and design spectra have made seismic evaluation a crucial topic of study. Adhitama et al. (2022) conducted a seismic assessment of an existing multi-story building using ASCE 41-17; however, their analysis primarily focused on performance classification and a Tier 3 nonlinear static analysis (pushover) using SAP2000. In contrast, this study comprehensively distinguishes itself by implementing ASCE 41-17 across Tiers 1–3. This approach provides a more detailed assessment of structural vulnerabilities and necessary retrofitting measures [2].

Speicher et al. (2020) applied ASCE 41-17 to a two-story cold-formed steel (CFS) building in a high seismic demand area. Their study evaluated the existing design and explored retrofitting measures necessary for compliance with the ASCE 41-17 standard. Despite the structure's compliance with ASCE 7 and AISI S400 and its successful performance in shake table tests beyond maximum considered earthquake levels, ASCE 41-17 still identified deficiencies. The challenges in applying ASCE 41-17 to cold-formed steel structures stem from differences in system overstrength and ductility. Their findings underscore the need to refine ASCE 41-17 to better account for full-system performance, thereby reinforcing its role as a benchmark performance-based seismic assessment standard [1].

In 2023, Indonesia experienced 10,789 earthquakes, a considerable increase over the yearly average of 7,000 occurrences. Of these, 861 were felt, and 24 caused damage; fortunately, no fatalities occurred. With its dense population and over 3,400 buildings, Jakarta faces significant seismic risks, as evidenced by the magnitude 6.7 earthquake in January 2022 [5][6]. The vulnerability of buildings constructed under outdated codes remains a pressing concern, despite the advancement of Indonesia's seismic legislation, with the most recent standard being SNI 1726:2019, which addresses the increasing seismic intensity [7, 8, 9]. Seismic examinations that follow international standards, such as ASCE 41-17, are critical for ensuring the structural integrity and safety of existing structures [10].

Various techniques are available for seismic assessments, including non-destructive testing, dynamic analysis, and static analysis [11]. Dynamic analysis models real-time ground motion interactions, capturing the complex mass and stiffness behaviors of structural systems, making it valuable for understanding responses under seismic conditions. On the other hand, static analysis applies equivalent static forces to evaluate structural responses, making it more suitable for more superficial structures and initial assessments. Together, these methodologies provide a comprehensive framework for analyzing and enhancing the structural resilience of earthquake-prone buildings [12]. Non-destructive testing evaluates structural and material characteristics without causing damage; it is crucial for aging structures commonly affected by poor soil conditions, pollution, or previous seismic occurrences, particularly in places like Jakarta [13][14]. Comprehensive evaluations and retrofitting measures are increasingly necessary to address vulnerabilities stemming from outdated designs and subpar construction standards [15].

ASCE 41-17, published by the American Society of Civil Engineers, provides a structured framework for assessing and retrofitting existing buildings to improve seismic resilience [16]. Its tiered methodology is integral to evaluating the performance and resilience of structures under seismic hazards. In Tier 1, visual inspections and basic calculations are performed to identify deficiencies and establish performance objectives aligned with the Basic Performance Objective for Existing Buildings (BPOE). Seismic hazards such as BSE-1E and BSE-2E are classified, with structural systems and material properties delineated for evaluation [17].

Tier 2 involves a more comprehensive evaluation to address the deficiencies identified in Tier 1. This phase uses linear static or dynamic analysis. While linear static analysis is applicable in simplified situations, linear dynamic analysis considers characteristics like elastic stiffness and viscous damping, resulting in a more realistic understanding of the building's response to seismic forces.

If inadequacies persist after Tier 2, the process advances to Tier 3, which includes advanced nonlinear analyses, such as pushover and time-history analysis. This tier thoroughly assesses structural performance under extreme seismic circumstances and the influence of suggested reinforcements. Finally, it ensures that all possible vulnerabilities are adequately addressed.

This systematic method facilitates the establishment of targeted retrofitting techniques, thereby improving structural resilience in seismically active zones.
By integrating ASCE, this systematic method facilitates the establishment of targeted retrofitting techniques, thereby improving structural resilience in seismically active zones. This study enhances safety and structural performance by integrating ASCE 41-17 standards with local seismic conditions, ensuring the durability and robustness of vital infrastructure.

## METHOD

This research methodology provides a comprehensive framework for evaluating the seismic performance of an eight-story office building in accordance with the ASCE 41-17 standard. The process begins with problem formulation, followed by a literature review and data collection to establish the foundational inputs for analysis. Building surveys, material testing, and on-site inspections are methods used to gather primary data, assessing the building's structural integrity and physical state. As-built drawings, old seismic records, and geotechnical reports are secondary data that can be analyzed to gain a better understanding of site-specific circumstances.

The seismic analysis progresses through a multi-tiered evaluation approach, as displayed in Figure 1. In Tier 1, an initial screening is conducted using ASCE 41-17 checklists to identify deficiencies such as soft stories or overturning instability. If deficiencies are found, Tier 2 is initiated, where a detailed linear analysis is performed using Modeling software. This phase applies deformation-controlled and force-controlled criteria to evaluate structural components, with capacity calculations verified against ASCE 41-17 acceptance criteria (M-factors). If Tier 2 analysis still identifies unresolved deficiencies, the study proceeds to Tier 3. In this final tier, advanced nonlinear analysis methods, including pushover and time-history simulations, are utilized to determine the ultimate capacity of the structure and provide insight into its failure mechanisms.

Figure 1 presents the complete flowchart outlining the sequence of methodological steps followed in this study.

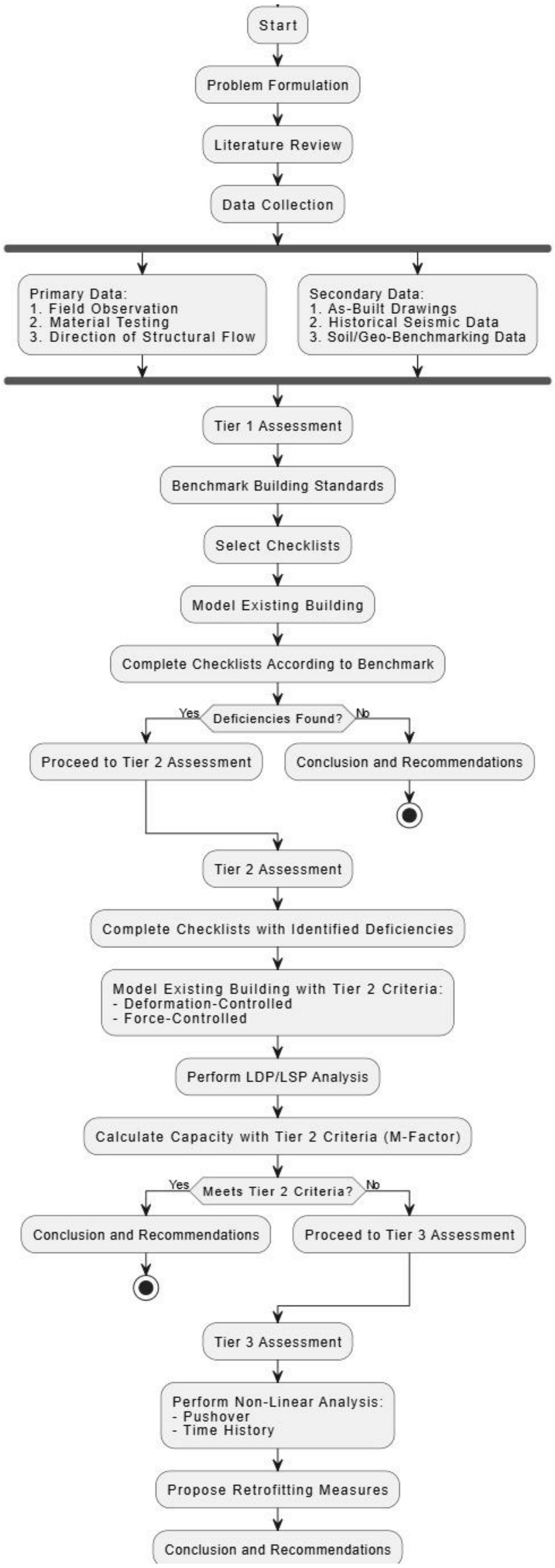

Figure 1. Flowchart of Research

## Data

This research employs a structured method to assess the seismic performance of an eight-story office building in Jakarta using the ASCE 41-17 standard. The process begins with problem formulation, followed by a literature review and data collection to establish foundational inputs for the analysis. The examined building was built in Kuningan, Jakarta, in 1989. Its structural system employs a Special Moment Resisting Frame (SMF) primarily consisting of reinforced concrete. It has eight floors, an additional MEP floor, and elevations ranging from ±0.00 m (Ground Floor) to +37.1 m (MEP Floor). The building geometry and model are illustrated in Figure 2, Figure 3, Figure 4, Figure 5, and Figure 6**Error! Reference source not found.**. The building is located at geographic coordinates -6.212681691672681, 106.83088139600952 (latitude and longitude), positioning it within a high-seismic-risk zone. The structural and material properties are as follows:

1. Concrete strength (fc') is 21 MPa (as determined by material testing).
2. Reinforcement Strength (fy):
   Diameter ≥ 12 mm: BJTD 39 or 390 MPa.
   Diameter < 12 mm: BJTP 24 or 240 MPa.

Detailed structural dimensions and concrete strengths of columns, beams, slabs, and shear walls are presented in Table 1, Table 2, Table 3, and Table 4.

These data were obtained from as-built drawings and verified through material testing. They serve as the basis for modeling in Software, applying load combinations, and conducting the seismic performance evaluation under Tier 1 screening.

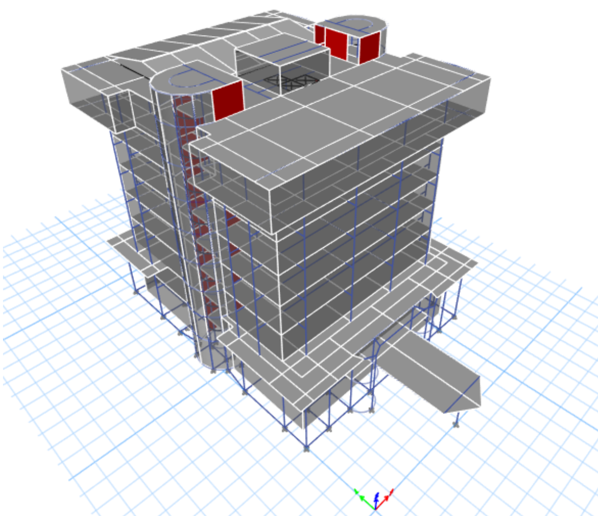

Figure 2. 3D Model of Existing Building

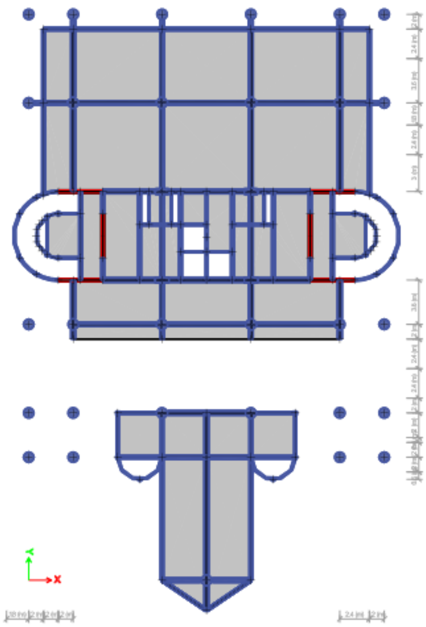

Figure 3. First floor of the existing building model.

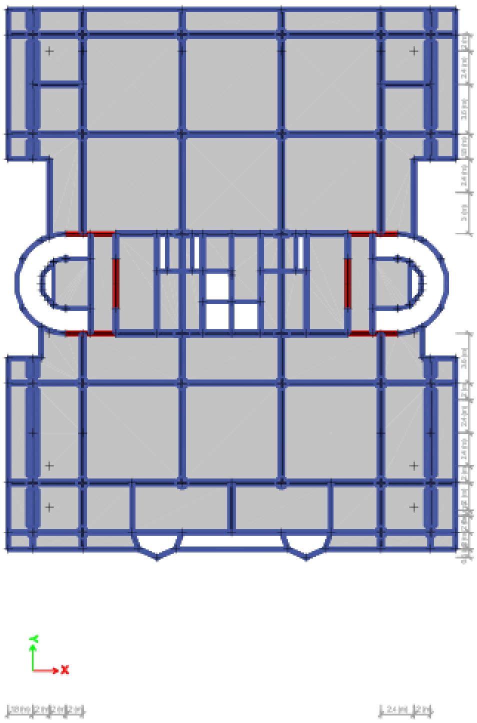

Figure 4. Second floor of the existing building model

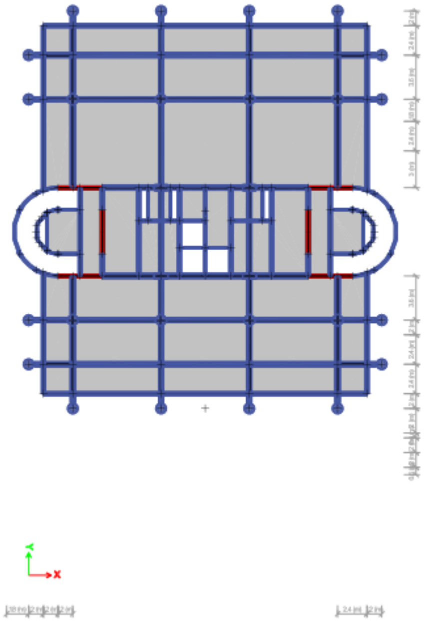

Figure 5. 3rd to 7th floor plan of the existing building model.

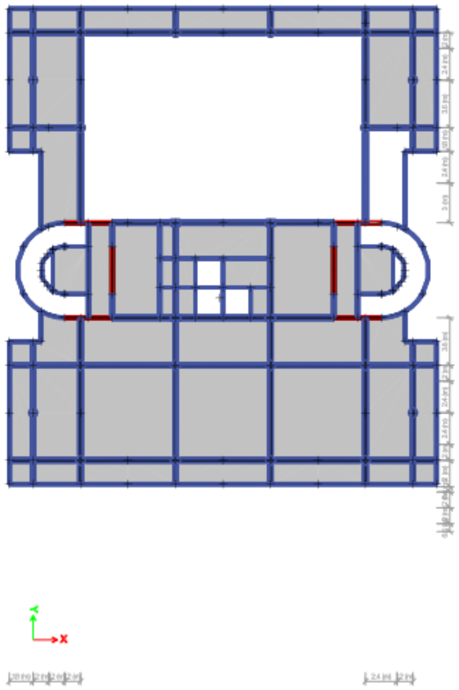

Figure 6. 3rd to 7th floor plan of the existing building model.

Table 1. Dimensions and Concrete Strength of Columns

| Column Type | Width (mm) | Height (mm) | Concrete strength (fc') |
|---|---|---|---|
| C1 | 900 | 900 | 21 MPa |
| C2 | 700 | 700 | 21 MPa |
| C3 | 500 | 1100 | 21 MPa |
| C4 | 500 | 750 | 21 MPa |
| C5 | 400 | 400 | 21 MPa |
| C6 | 260 | 500 | 21 MPa |
| C7 | 300 | 300 | 21 MPa |

Table 2. Dimensions and Concrete Strength of Beams

| Beam Type | Width (mm) | Height (mm) | Concrete strength (fc') |
|---|---|---|---|
| B1 | 250 | 400 | 21 MPa |
| B2 | 300 | 500 | 21 MPa |
| B3 | 400 | 600 | 21 MPa |
| B4 | 500 | 500 | 21 MPa |
| B5 | 250 | 300 | 21 MPa |
| B6 | 300 | 600 | 21 MPa |
| B7 | 400 | 700 | 21 MPa |
| B8 | 900 | 1000 | 21 MPa |

Table 3. Dimensions and Concrete Strength of Slabs

| Slab Type | Thickness (mm) | Concrete strength (fc') | Slab Type |
|---|---|---|---|
| S1 | 150 | 21 MPa | S1 |
| S2 | 120 | 21 MPa | S2 |
| R1 | 150 | 21 MPa | R1 |
| R2 | 120 | 21 MPa | R2 |

Table 4. Dimensions and Concrete Strength of Shear Wall

| Shear Wall Type | Thickness (mm) | Concrete strength (fc') | Shear Wall Type |
|---|---|---|---|
| S | 150 | 21 MPa | S1 |

The research method incorporates a transparent research model, represented by a flowchart, which sequentially illustrates data collection, analysis, and evaluation steps. This structured approach is underpinned by theory development, integrating principles from ASCE 41-17 and SNI 1726:2019 standards to ensure compliance with international and local safety benchmarks. The study makes a scientific contribution by enhancing existing seismic assessment methodologies through the systematic application of tiered evaluations and by proposing retrofitting measures based on the findings. Moreover, this methodology facilitates the establishment of a performance-based seismic design framework aimed at improving structural resilience in areas prone to high seismic activity. By integrating theoretical analysis, model formulation, and real-world applications, this study makes a substantial contribution to the field of structural engineering. It combines established theories and improvises on current assessment methods by tailoring them to site-specific conditions and modern software capabilities. The methodology ultimately offers a replicable framework for future seismic evaluations and retrofitting strategies, addressing a critical gap in the resilience of older buildings against seismic threats.

## RESULTS AND DISCUSSION

### Tier 1 Evaluation

The Tier 1 evaluation process begins with a structured checklist tailored to the specific characteristics of the building under review, as referenced in Table 5. This checklist is a valuable tool for carefully analyzing various structural components and situations, guaranteeing that all crucial aspects of seismic behavior are thoroughly investigated. The results of this study classify the building's status into four categories: unknown, non-compliant, compliant, and not applicable.

These categorizations help determine which areas require additional investigation and involvement, as shown in Table 6. For example, a "compliant" status implies that the building meets the seismic performance requirements. In contrast, a "non-compliant" rating identifies flaws that must be corrected in the following levels of analysis.

The "not applicable" status may apply to architectural features unrelated to the seismic assessment, whereas "unknown" refers to places where data is insufficient or ambiguous.

This systematic evaluation approach helps determine the building's initial seismic resistance and informs later assessments required for retrofitting and improving overall structural performance.

Table 5. Benchmark for Existing Building Tier 1

| Category | Details |
|---|---|
| Building Risk Categories | Risk Category II |
| Performance Levels | BSE-2E, Collapse Prevention (CP) |
| Seismicity Levels | Ss = 0.8032g and Sd1 = 0.3894g, High Seismicity |
| Checklist Selection | Basic Configuration Checklist, Structural Checklist for Collapse Prevention Performance |

Source: Adapted from ASCE 41-17, Table 2-2, p. 71 (ASCE, 2017).

Table 6. Basic Configuration Checklist

| No | Item Evaluated | Status |
|---|---|---|
| 1 | Load Path | C |
| 2 | Adjacent Buildings | C |
| 3 | Mezzanine | C |
| 4 | Weak Story | C |
| 5 | Soft Story | NC |
| 6 | Vertical Irregularity | NC |
| 7 | Geometry | NC |
| 8 | Mass | NC |
| 9 | Torsion | C |
| 10 | Liquefaction | C |
| 11 | Slope Failure | C |
| 12 | Surface Fault Rupture | U |
| 13 | Racking | NC |
| 14 | Foundation Tie-down | C |

Source: Adapted from ASCE 41-17, Table 17-2, p. 315 (ASCE, 2017)

The Tier 1 assessment of Collapse Prevention (CP) performance under BPOE BSE-2E classification for the Existing Building demonstrates serious noncompliance with ASCE 41-17 requirements. Based on Table 6, five of the 14 configuration checklist elements reviewed were non-compliant (NC), including soft story, vertical irregularity, geometry, mass, and racking. In contrast, one item, surface fault rupture, was marked as unknown. The remaining eight issues were compliant (C), including the load path, adjacent buildings, mezzanine, torsion, liquefaction, slope failure, and foundation ties. These data suggest that the structure warrants additional study via a Tier 2 assessment.

Table 7. Collapse Prevention Structural Checklist for Concrete Frame (C1)

| No | Item Evaluated | Status |
|---|---|---|
| 1 | Redundancy | C |
| 2 | Column Axial Stress | NC |
| 3 | Concrete Columns | C |
| 4 | Infill Walls | C |
| 5 | Column Shear Stress | NC |
| 6 | Flat Plate Frame | C |
| 7 | Pre-tensioned Frame Elements | C |
| 8 | Floating Columns | C |
| 9 | Shear Failure | NC |
| 10 | Substantial Column - Weak Beam | NC |
| 11 | Beam Reinforcement | C |
| 12 | Column Reinforcement Splice | C |
| 13 | Beam Reinforcement Splice | C |
| 14 | Column Tie Spacing | NC |
| 15 | Beam Tie Spacing | C |
| 16 | Deflection Compatibility | C |
| 17 | Flat Plate | C |
| 18 | Diaphragm Continuity | C |
| 19 | Uplift Resistance in Poer/Pile Cap | C |

Source: Adapted from ASCE 41-17, Table 17-22, p. 343 (ASCE, 2017).

Referring to Table 7, the assessment of the existing building's CP performance for C1 (Moment Frames) under ASCE 41-17 identified five out of 19 evaluated items as non-compliant (NC). The primary reasons for noncompliance were insufficient material strength and inadequate column detailing, which rendered the columns incapable of withstanding shear stresses induced by seismic forces. The findings emphasize that improper stirrup placement could contribute to column failure. Likewise, Table 7 indicates that several structural components did not satisfy shear capacity requirements due to deficiencies in beam-column joint detailing and inadequate stirrup configuration.

For C2 (Shear Walls), 3 out of 7 evaluated items were classified as compliant (C), while one item was marked as unknown (U) due to missing uplift pile cap details in **Error! Not a valid bookmark self-reference.**. The remaining three items were categorized as not applicable (N/A). These findings indicate that although most shear wall components satisfy CP standards, further investigation is necessary for elements classified as unknown or non-relevant to ensure comprehensive seismic performance.

## Tier 2 Evaluation

The Tier 2 assessment utilizes seismic force scaling with modification factors in accordance with ASCE 41-17 guidelines. These parameters help determine the demand and force conditions acting on structural elements, such as beams, columns, joints, and shear walls, as shown in Table 6, Table 7, and Table 8. This approach ensures that the evaluation appropriately represents the seismic performance and resilience of the structure under projected seismic stress in Table 9 [16].

Table 8. Collapse Prevention Structural Checklist for Shearwall Frame (C2)

| No | Item Evaluated | Status |
|---|---|---|
| 1 | Shear Stress Check | C |
| 3 | Shear Wall Anchors | N/A |
| 4 | Load Transfer to Shear Wall | C |
| 5 | Coupling Beams | N/A |
| 6 | Diaphragm Continuity | C |
| 7 | Openings in Shear Walls | N/A |
| 8 | Uplift Pile Cap | U |

Source: Adapted from ASCE 41-17, Table 17-24, p. 346 (ASCE, 2017)

Table 9. Seismic Force Calculation Tier 2

| Direction | Spectrum Response |
|---|---|
| X | 16077.77 |
| Y | 15118.16 |

Table 9 displays the seismic force calculations obtained during the Tier 2 evaluation, showing spectral response values of 16,077.77 in the X direction and 15,118.16 in the Y direction. These values are derived from the short-period spectral response (Ss) and the 1-second spectral response (Sd1), scaled according to the site-specific seismic characteristics using Response Spectrum Analysis (RSA). This analysis is crucial for determining the lateral forces that the structure must withstand during seismic events, thereby ensuring an adequate design for safety and stability [3][17].

The classification of structural component actions used in the evaluation is provided in Table 10, based on ASCE 41-17. Table 11 illustrates the relationship between DCR values and the ductility standards outlined in ASCE 41-17.

The capacity of structural elements in Table 10 is analyzed using m-factors based on the type of component action. Beams, which use Deformation-Controlled Action, require m-factors that vary according to the ratio of ρ−ρ′/ρbal and transverse reinforcement. Columns and joints that use force-controlled action do not require m-factors.

Table 10. Classification of Structural Component Actions

| Components | Deformation-Controlled Action | Force-Controlled Action |
|---|---|---|
| Momen Frame | | |
| • Beam | Moment (M) | (V) |
| • Column | — | P, V |
| • Joints | | V |
| Shear Wall | M, V | P |
| Bracing Frame | | |
| • Bracing | P | — |
| • Beam | — | P |
| • Column | — | P |
| • Shear link | V | P, M |
| Connections | P, V, Mb | P, V, M |
| Diaphragms | M, Vc | P, V, M |

Source: Adapted from ASCE 41-17, Table C7-1, p. 485 (ASCE, 2017)

Table 11. Relationship Between DCR Value and Ductility Requirements

| Maximum DCR or Displacement Ductility | Description |
|---|---|
| < 2 | Low ductility requirements |
| 2- 4 | Medium ductility requirements |
| > 4 | High ductility requirements |

Source: ASCE 41-17

For shear walls, which also utilize Deformation-Controlled Action, m-factors are determined based on the confining boundary and component type, with different values for IO, LS, and CP conditions. All references are based on ASCE 41-17 [18].

Table 11 illustrates the relationship between DCR values and the ductility standards outlined in ASCE 41-17. The DCR is a crucial indicator of a structure's seismic performance, demonstrating its ability to withstand deformation without sustaining severe damage.

DCR < 2. It suggests they are designed to withstand moderate deformation while maintaining structural integrity. This level of ductility is often considered adequate for areas with moderate seismic risk, where some damage may occur but collapse is not expected.

DCR 2-4: Structures have medium ductility requirements in this range. It implies that they are expected to accommodate moderate levels of deformation while still maintaining structural integrity. This level of ductility is often deemed appropriate for areas with moderate seismic risk, where some damage may occur, but collapse is not anticipated.

DCR > 4: A DCR exceeding 4 signifies high ductility requirements. Structures in this category are designed to withstand significant deformations during seismic events, allowing them to disperse energy and avoid catastrophic failure efficiently. This design approach is critical for buildings in seismically active zones with potential for significant ground movement [18].

The Tier 2 assessment results indicate that several structural elements of the building require special attention, particularly those with high ductility demands. Beams, columns, joints, and shear walls with high Demand Capacity Ratios (DCR) are at significant risk of failure during an earthquake (Table 12, Table 13, Table 14, and Table 15). Therefore, additional reinforcement is necessary to increase ductility capacity and reduce the risk of collapse [19].

Table 12. Beam Ductility Requirements Categories

| Beam Type | Ductility Requirements | DCR |
|---|---|---|
| B 20x40 | High ductility requirements | 9.357 |
| B 30x40 | High ductility requirements | 7.241 |
| B 30x50 | High ductility requirements | 9.459 |
| B 90x1000 | High ductility requirements | 7.640 |
| B 40x60 | Medium ductility requirements | 3.518 |
| B 40x70 | Medium ductility requirements | 2.605 |
| B 25x40 | Low ductility requirements | 1.026 |

Table 13. Column Ductility Requirements Categories

| Column Type | Ductility Requirements | DCR |
|---|---|---|
| C 50 x 110 | High ductility requirements | 5.894 |
| C 90 | High ductility requirements | 4.909 |
| C 50 x 75 | Medium ductility requirements | 2.515 |
| C 40 x 40 | Medium ductility requirements | 2.342 |
| C 26 x 50 | Medium ductility requirements | 3.400 |
| C 70 | Low ductility requirements | 0.950 |
| C 30 x 30 | Low ductility requirements | 0.298 |

Table 14. Joint Ductility Requirements Categories

| Joint | Ductility Requirements | Nilai DCR |
|---|---|---|
| C 50 x 110 | Low ductility requirements | 1.620 |
| C 90 | Low ductility requirements | 1.092 |
| C 70 | Low ductility requirements | 0.888 |
| C 50 x 75 | Low ductility requirements | 0.926 |
| C 40 x 40 | Low ductility requirements | 0.643 |
| C 26 x 50 | Low ductility requirements | 1.133 |
| C 30 x 30 | Low ductility requirements | 0.841 |

Table 15. Shear Wall Ductility Requirements Categories

| Shear Wall | Ductility Requirements | Nilai DCR |
|---|---|---|
| W1=W3=W4=W6 | Low ductility requirements | 1.402 |
| W2=W5 | Low ductility requirements | 1.125 |

Beams such as B 20x40, B 30x40, B 30x50, and B 90x1000, which have high ductility demands, require immediate intervention to ensure structural safety, as shown in Table 12. Columns with high DCR values, such as C 50x110 and C 90, also require reinforcement to maintain the structure's stability, as shown in Table 13.

Furthermore, joints with low ductility demands in Table 14, such as C 50x110 and C 26x50, may become weak points in the structure if not reinforced. Shear walls in Table 15, categorized as low ductility, also require further evaluation and reinforcement to withstand significant lateral forces during an earthquake [20].

## Tier 3 Evaluation

In Tier 3, a nonlinear analysis, specifically a pushover analysis, was conducted. The pushover curve represents the relationship between base shear force (V) and displacement at a reference point (δ) [21]. The analysis was performed in each orthogonal direction of the Existing Building using the following displacement control parameters:

1. Magnitude = 742 mm (2% of the total building height, 37,100 mm), meeting 150% of the displacement target.
2. Joint = Topmost node at the end of the concrete structure (Top Floor).

The moment-curvature curve of the beam is derived from the analysis results of the section designer in Modeling Software, which is simplified to align with the available moment-curvature column in the program. Meanwhile, the plastic shear hinge utilizes Shear V2—the pushover analysis results in Figure 7.

Displacement tends to increase from the lower floors to the topmost floor, with the highest value recorded at the R. MEP roof. In the X direction, the maximum displacement is 59.104 mm at the R. MEP roof, while in the Y direction, the highest value is 59.297 mm at the same level. This increase in displacement indicates the accumulation of loads and forces acting more significantly on the upper floors [22]. The difference in values between the X and Y directions reflects variations in the structure's response to lateral loads that show in Figure 8 [23].

According to ASCE 41-17, the target displacements for structures should align with the limits specified in FEMA 356 [9][24]. These limits dictate that the maximum displacement for the Immediate Occupancy (IO) performance level is 1%, the Life Safety (LS) level is 2%, and the Collapse Prevention (CP) level is 4%.

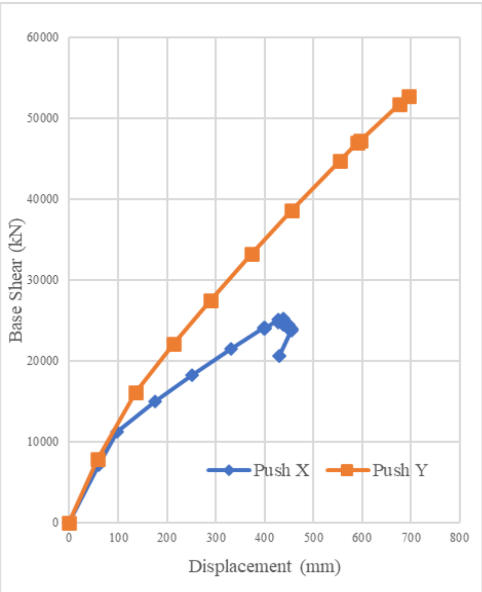


Figure 7. Story Displacement Base Shear

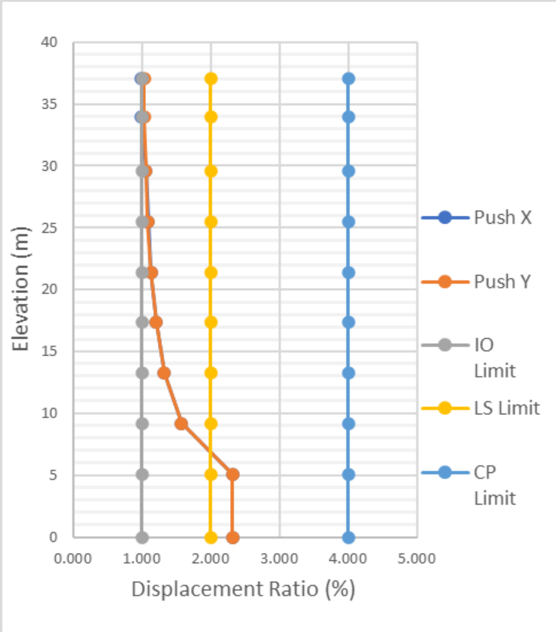


Figure 8. Story Displacement Ratio

The most considerable inter-story drift recorded is 2.321% in the X direction and 2.319% in the Y direction on the first floor (Figure 8). These values exceed the 2% maximum limit set for the Life Safety (LS) performance level, indicating that the building is nearing the Collapse Prevention (CP) threshold at this level. It suggests that the building operates between the Life Safety (LS) and Collapse Prevention (CP) performance levels, particularly on the lower floors. While the structure still meets the Life Safety standards and prevents total collapse, the increased deformation at the lower floors indicates that special attention is needed to ensure safety and stability under more extreme loading conditions.

Based on the running output, one element reached the yielding condition (B-C) at step 1. As the steps progress, the number of elements reaching yielding conditions increases. By the final step, in the X direction, 413 elements experienced yielding (Figure 9 and Table 16) in step 36. In the pushover analysis in the Y direction, 11 steps are observed, with six elements yielding first at step 1 and 528 elements reaching yielding conditions by step 11, as shown in Figure 10 and Table 17.

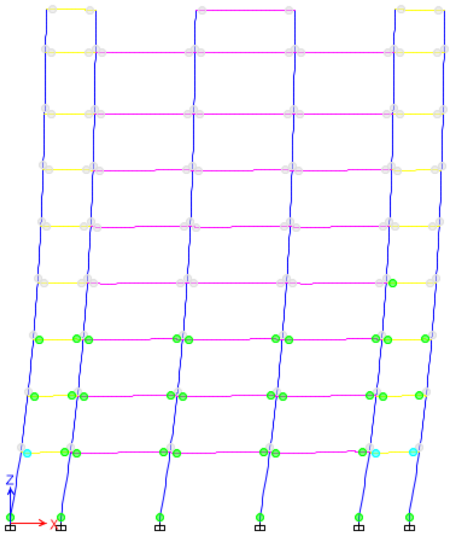

Figure 9. Location of Plastic Hinges in the X Direction

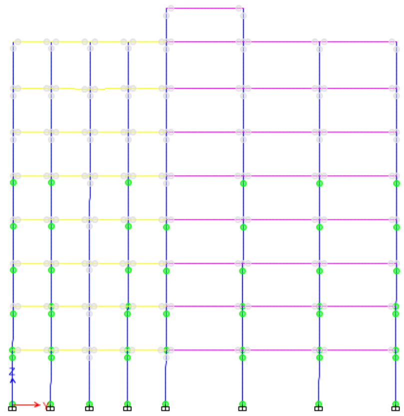

Figure 10. Location of Plastic Hinges in the Y Direction

Table 16. Distribution of Plastic Hinges in the X Direction

| Step | Monitored Displ | Base Force | A-IO | IO-LS | LS-CP | >CP | Total |
|---|---|---|---|---|---|---|---|
| | mm | kN | | | | | |
| 0 | 0 | 0 | 1476 | 0 | 0 | 0 | 1476 |
| 1 | 59.10 | 7180.07 | 1476 | 0 | 0 | 0 | 1476 |
| 2 | 97.86 | 11238.64 | 1476 | 0 | 0 | 0 | 1476 |
| 3 | 174.92 | 15094.92 | 1434 | 42 | 0 | 0 | 1476 |
| 4 | 251.88 | 18334.88 | 1372 | 104 | 0 | 0 | 1476 |
| 5 | 331.06 | 21528.15 | 1294 | 182 | 0 | 0 | 1476 |
| 36 | 429.76 | 20711.39 | 1239 | 223 | 14 | 0 | 1476 |

Table 17. Distribution of Plastic Hinges in the Y Direction

| Step | Monitored Displ | Base Force | A-IO | IO-LS | LS-CP | >CP | Total |
|---|---|---|---|---|---|---|---|
| | mm | kN | | | | | |
| 0 | 0 | 0 | 1476 | 0 | 0 | 0 | 1476 |
| 1 | 58.25 | 7937.39 | 1476 | 0 | 0 | 0 | 1476 |
| 2 | 135.09 | 16120.77 | 1476 | 0 | 0 | 0 | 1476 |
| 3 | 213.01 | 22209.20 | 1474 | 2 | 0 | 0 | 1476 |
| 4 | 289.38 | 27597.50 | 1448 | 28 | 0 | 0 | 1476 |
| 5 | 374.23 | 33316.77 | 1366 | 110 | 0 | 0 | 1476 |
| 11 | 694.51 | 52846.64 | 1110 | 318 | 48 | 0 | 1476 |

A building structure's "strong column-weak beam" requirement is satisfied if plastic hinges form first in the beams [25]. According to Figures 4 and 5, the most critical elements in this study are located in the columns with Life Safety (LS) performance in both X and Y directions. This condition leads to the loss of the building's ability to maintain a "continuity strong mechanism," which involves the formation of plastic hinges in the columns at both the top and bottom of the vertical structure [26]. In this study, plastic hinges in the columns occur due to the suboptimal design of 900 mm columns, which are inadequate to support the load. Therefore, beams and columns should ensure sufficient dimensions to support the loads effectively.

According to the ATC-40 methodology [27], Table 18 outlines the roof drift ratio limitations, categorizing structural performance levels based on the maximum total drift and the maximum total inelastic drift.

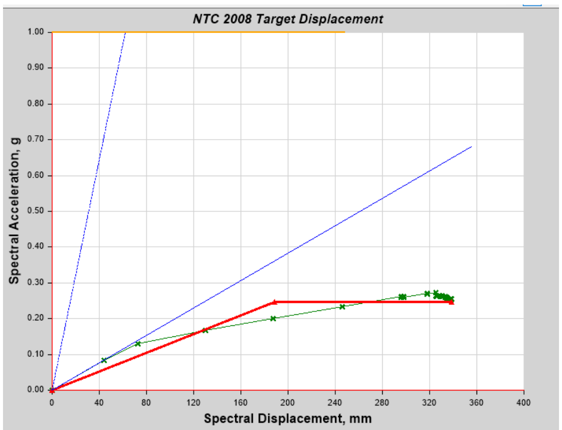


Figure 11. Structural Performance According to ATC-40 Push X

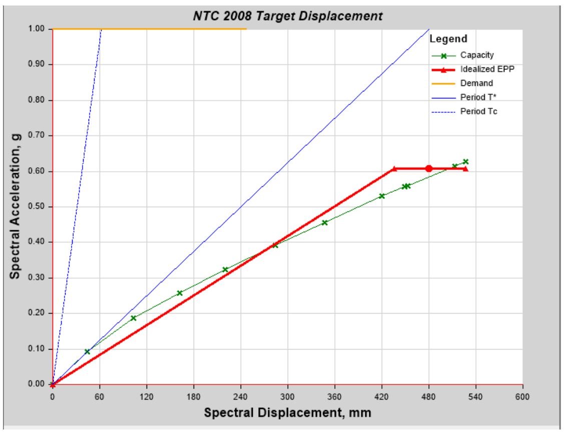


Figure 12. Structural Performance According to ATC-40 Push Y

Table 18. Roof drift ratio limitation according to ATC-40

| Parameter | Performance Level | | | |
|---|---|---|---|---|
| | IO | DC | LS | SS |
| Maximum Total Drift | 0,01 | 0,01 s/d 0,02 | 0,02 | $0,33\frac{V_i}{P_i}$ |
| Maximum Total Inelastic Drift | 0,005 | 0,005 s/d 0,015 | No Limit | No Limit |

Source: ATC-40

Table 19. Structural Performance Levels

| Direction | Displacement (mm) | Maximum Total Drift | Performance Level |
|---|---|---|---|
| X | 523.22 | 0.0141 | Damage Control |
| Y | 631.462 | 0.0170 | Damage Control |

Source: Author

The Damage Control (DC) level is defined by a maximum total drift between 0.01 and 0.02 and an allowable maximum total inelastic drift between 0.005 and 0.015. Structures classified under DC can withstand significant seismic forces but may experience moderate damage that affects functionality, although they are not at immediate risk of collapse.

Table 19 presents the Modeling Software simulation results, showing that the structure exhibits a maximum total drift of 0.0141 in the X-direction and 0.0170 in the Y-direction, both of which fall within the Damage Control (DC) performance level based on ATC-40 criteria (Figure 11 and Figure 12). This classification indicates that while the building remains stable during seismic events, it may sustain structural and non-structural damage that could impair its usability without intervention.

Although the structure passes Damage Control (DC) requirements, reinforcing actions are recommended to improve long-term stability and safety, particularly in seismically active areas. Numerous seismic evaluation frameworks emphasize the importance of proactive retrofitting to minimize potential damage and ensure post-earthquake performance.

One crucial factor is whether the structure can be adapted to meet Immediate Occupancy (IO) requirements, particularly in essential load-bearing components such as columns, beams, and shear walls. Achieving IO performance allows the building to continue functioning immediately following an earthquake, decreasing downtime and repair expenses. Reinforcing measures should be implemented even if the structure meets the Damage Control standards. Such enhancements provide long-term stability and safety, particularly in seismically active areas. Numerous seismic assessment frameworks emphasize the importance of retrofitting, advocating for proactive improvements to structural integrity to mitigate hazards associated with future seismic events. By implementing these steps, building owners can significantly reduce the likelihood of catastrophic damage, thereby strengthening the structure's overall resilience [28].

Pushover analysis shows that the Existing Building is at risk of failing to meet the Collapse Prevention (CP) performance level, even if the overall drift is within the Damage Control (DC) limits. It is significant for structural components that may experience severe deformation in the later stages of the study, which could jeopardize the building's seismic performance. Addressing these vulnerabilities with proper retrofitting solutions is vital for improving safety and resilience against seismic disasters [29].

The existing building does not meet the Collapse Prevention (CP) performance criteria for many reasons. Although the building's elastic design tries to reduce structural damage and assure occupant safety, this strategy cannot meet the CP performance requirement. While the elastic design allows the building to endure deformation without significant permanent damage, it may lack the rigidity to conform to the minimal deformation requirements established by CP performance standards.

Research highlights that relying solely on elastic design strategies can lead to an insufficient structural response during severe seismic events, potentially exposing the building to risks of failure or excessive damage [30]. Effective seismic design must prioritize both the ability to withstand deformation and the necessity for rigidity to meet stringent performance objectives, particularly in seismically active regions [31]. Thus, enhancing structural integrity through retrofitting or adopting more resilient design principles becomes imperative to ensure compliance with performance criteria in future seismic assessments [32].

Relatively small structural element sizes were selected to reduce costs and material waste [33]. However, this also impacts the rigidity and strength of the structure. Smaller components typically have lower lateral load resistance, which makes it harder for the building to achieve the Collapse Prevention (CP) performance target [34]. Additionally, the lack of lateral stiffness affects the building's overall performance in meeting the Collapse Prevention (CP) requirement, even though columns provide significant structural support and have an average height on each floor [35]. Variations in structural element capacity can also be caused by differences in reinforcement and dimensional rounding between the original design and field execution [36]. These differences may cause structural components to be less capable of withstanding earthquake loads, failing to

As noted by prior studies, structural elements were designed with relatively small dimensions to optimize cost and minimize material waste [37]. However, this choice compromises the overall structure's stiffness and strength. Typically, smaller components exhibit reduced lateral load resistance, posing challenges for the building to meet the Collapse Prevention (CP) performance criteria [38]. Insufficient lateral stiffness adversely affects the building's ability to satisfy the CP requirement despite the columns providing considerable structural support with a standard height across each floor level [34]. Furthermore, disparities in the structural capacity of elements may arise due to variations in reinforcement placement and dimensional adjustments made during field implementation, as opposed to the initial design specifications [39]. Such discrepancies can reduce the structural elements' resilience against seismic forces, potentially compromising their earthquake-resistant capacity.

Considering all variables, the Existing Building does not fulfill the Collapse Avoidance (CP) measures in ASCE 41-17. A more comprehensive approach is required during the planning and development stages to enhance its resistance to strong seismic tremors. Level 1 to Level 3 evaluations reveal that the basic components, columns, and pillars are insufficient in meeting assembly CP criteria. Reinforcing these components with fiber-reinforced polymer (FRP) is recommended to enhance their capacity and performance when withstanding seismic loads.

The ASCE 41-17 standard supports the use of Fiber-Reinforced Polymer (FRP) reinforcing as a viable method to enhance the shear and flexural capacities of auxiliary elements, such as bars and columns, while minimizing the additional dead load. Applying FRP to these surfaces enhances their ductility, mitigates the risk of splitting or failure under seismic loading, and improves their lateral strength. This approach is crucial for enhancing the strength and resilience of structures in seismic-prone zones, ensuring compliance with advanced design safety standards [40].

Applying fiber-reinforced polymer (FRP) to columns enhances their capacity to resist hub and shear stresses, effectively preventing collapse due to localized stresses. This fortifying method enhances the columns' overall structural integrity, enabling them to withstand greater loads and resist damage during seismic events. By fortifying these basic components, engineers can enhance the strength of structures in seismic-prone areas, ensuring they meet safety standards and performance requirements. [41]. Integrating fiber-reinforced polymeric materials (FLPs) into structural components significantly improves performance under various stress conditions. FRP enhances the axial and seismic capacity of a column, thereby reducing the risk of collapse due to concentrated stresses. This improvement is significant to ensure the structural integrity of columns exposed to high loads and dynamic forces. [42]

Additionally, applying FRP to pillars enhances the flexural and shear properties, allowing these components to experience more pronounced distortion without encountering critical damage. Its progressive distortion capacity plays a vital role in enhancing overall flexibility during seismic events, thereby reducing the likelihood of failure. Ponders have illustrated that fortifying reinforced concrete (RC) bars with FRP increases their load-bearing capacity and resistance to breaking. Additionally, FRP provides a strong and lightweight alternative to traditional support procedures, making it a successful solution for reinforcing structures in seismic-prone areas. [43].

Stirrups are prompted to progress in sidelong imprisonment and columns' execution during seismic events. This approach increases the columns' capacity to resist seismic stresses without significant damage. Moreover, fortifying beam-column joints is essential, as these connections are often vulnerable during seismic tremors. The flexural and shear capacities can be improved by adding additional fortification at these intersections [44]. Besides, fortifying and hardening the floor stomach is essential to ensure superior horizontal push transmission to shear walls or other vertical primary components. This can be achieved by incorporating a reinforced concrete topping piece or applying an FRP layer to the diaphragm's surface. Such adjustments enhance the auxiliary judgment and provide support to meet the Collapse Anticipation (CP) criteria outlined in ASCE 41-17 [45]. Lastly, better lateral stress distribution to shear walls or other vertical elements can be ensured by strengthening and stiffening the floor diaphragm, for example, by installing a reinforced concrete topping slab or FRP layer on the diaphragm surface [46]. By implementing these reinforcing measures, the Existing Building is expected to achieve enhanced auxiliary security and stability during seismic events, ultimately improving its resilience and compliance with established performance standards.

## CONCLUSION

The NC (Non-Compliant) classification within the seismic assessment table indicates that the auxiliary elements fail to meet the desired seismic design criteria, rendering the building unfit to withstand seismic forces effectively. The Level 3 assessment assures that basic components are within the LS-Cp (Life Security - Collapse Avoidance) condition, implying that. In contrast, the structure may maintain its integrity during a seismic tremor, but it is at high risk of partial or complete collapse under more intense seismic loads.

Moreover, discoveries from Level 1 and Level 2 assessments show that hub push does not fulfill the endorsed necessities, illustrating deficient compressive capacity in key auxiliary components. The tall Demand-Capacity Proportion (DCR) values observed in bars, columns, and joints indicate considerable ductility requirements, which compromise the fundamental integrity of the building. The Level 3 analysis confirms that, with a versatile plan approach, the building fails to meet the CP (Collapse Anticipation) execution target as specified in ASCE 41-17. This highlights the need for auxiliary retrofitting to enhance the building's seismic resilience.

To address these insufficiencies, we propose the application of CFRP (carbon fiber-reinforced polymer) as a reinforcing strategy. Given its tall, malleable nature, CFRP can successfully enhance both pivotal capacity and shear resistance, thereby improving the overall seismic performance of the structure. It contributes to seismic appraisal by providing a comprehensive, multi-tier examination of a maturing structure, utilizing ASCE 41-17, and identifying fundamental shortcomings that require immediate attention. The discoveries highlight the limitations of routine plan approaches in older buildings and emphasize the need to focus on fortification techniques.

Future inquiries should coordinate structural appraisals to refine the precision of seismic execution forecasts. Moreover, advanced examinations ought to center on optimizing fortifying strategies, especially for fundamental load-bearing components such as columns, pillars, and shear walls, to improve the ductility and, by and enormous, seismic flexibility of existing structures

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