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# Structural Evaluation of a Low-Rise Steel Building in Jakarta

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**ABSTRACT**
Due to increasing awareness of human safety, Indonesia has mandated that all operational buildings obtain a Certificate of Occupancy. Consequently, existing structures must undergo a structural assessment before being granted this certificate. Evaluating older buildings poses significant challenges, especially when comprehensive records are lacking, and budgets do not allow for Non-Destructive Testing (NDT). This study presents the structural evaluation findings of a three-story steel building in Jakarta. Using visual inspections, field measurements, and limited available data, an analytical model representing the building's actual condition was developed. The evaluation followed three procedures: Tier 1 and Tier 2 evaluations of ASCE 41-17 and proportional seismic forces. These procedures aimed to gauge the structural integrity and identify areas vulnerable to failure during a severe earthquake. The assessment focused on the ductility of the seismic force-resisting system components and the strength of beam-column joint connections against specified acceptance criteria. The findings highlight critical insights into the building's structural performance, informing decisions on necessary measures such as structural reinforcement, occupancy restrictions, or demolition. This study underscores the importance of thorough structural assessments in ensuring the safety and resilience of older buildings in earthquake-prone regions.
**Keywords**: ASCE 41; Existing Building Evaluation; Proportional Seismic Forces; Steel Building.

## INTRODUCTION

Lately, structural evaluation work for the purpose of obtaining a Certificate of Occupancy has been increasingly performed in major cities and industrial areas across Indonesia (Thakkar, 2020). Various buildings and industrial structures that have reached the end of their service life require evaluation concerning their architectural function, structural strength, and MEP functions (Daoudi et al., 2019; Han et al., 2023; Wibowo et al., 2024).

Assessing the structures of existing buildings is not an easy task. Drawings and as-built reports containing information on structural systems, foundation systems, and material specifications are rarely available (Bitro et al., 2024; Caprani & Khan, 2024). If they do exist, they are often very limited. Moreover, due to budget and time constraints, most building owners reject proposals for various tests on the structural material strength of their buildings (Hareendran et al., 2023; Mertens et al., 2021). Therefore, structural assessors must keenly observe and extract information solely from what can be seen in the field (Miner-Romanoff, 2023; Papagiannopoulos et al., 2021). If fortunate, they may be allowed to dismantle a small portion of the ceiling to peek at the roof structure and conduct minor excavations to understand the foundation system. Otherwise, it's just visual observation.

The purpose of this research is to share the experience of conducting assessments on low-rise steel structure buildings located in Jakarta, with very limited data and without conducting any tests. The seismic analysis and evaluation were carried out based on ASCE 41-17 (awaiting its official version in SNI) and relevant Indonesian National Standards (SNI) for buildings, including SNI 1729:2020, SNI 7860:2020, SNI 7972:2020, SNI 1726:2019, and SNI 1727:2020 (Jauhari et al., 2021; Santoso & Astawa, 2022; Setiawan et al., 2021).

A three-story steel structure building owned by one of the private universities will represent the existing low-rise steel structure buildings in Jakarta, serving as a case study building in this paper (Becker et al., 2023; Kumar et al., 2024; Papagiannopoulos et al., 2021). Erected since the 1980s, the building is currently utilized as classrooms, offices, and a cafeteria. Apart from obtaining the SLF, the university has requested a structural evaluation for the potential replacement of the building facade and the conversion of rooms into a co-working space and a university medical center.

For facade alterations and room function conversions, structural analysis and evaluation are conducted based on the latest loading regulations (SNI 1727:2020) using Tier 1 and Tier 2 evaluations from ASCE 41-17, as well as the proportional seismic force method developed by the author's team (De Domenico et al., 2024; Khala et al., 2022; Wang et al., 2023).

It is important to note that this structural assessment is conducted solely based on data obtained from visual observations on-site, without conducting Destructive Tests (DT) or Non-Destructive Tests (NDT), and relying on construction knowledge from the 1980s, without any existing structural reports or drawings (Amin et al., 2019; Ding et al., 2024; Liu et al., 2021).

This study aims to provide insights into the structural assessment process under constraints of minimal data availability, relying on visual site observations and historical construction knowledge, without existing structural reports or drawings. The findings and methodologies discussed herein are intended to contribute to the body of knowledge on structural evaluation practices for low-rise buildings in seismic regions, particularly in developing countries where such limitations are common.

## RESEARCH METHODS

The research uses three methods to evaluate the seismic performance of existing buildings. It follows Tier 1 and Tier 2 evaluations from ASCE 41-17 standards. Tier 1 involves using a checklist covering structural integrity and earthquake resistance. Some checklist items can be checked visually, like spotting structural damage. But for others, calculations using ASCE 41-17 formulas are needed, like checking if load-bearing parts are strong enough against earthquakes.

After Tier 1 evaluation, Tier 2 evaluation is needed to evaluate any deficiencies identified in Tier 1. But Tier 2 has different criteria, needing a more detailed approach. Here, a mathematical model is used to analyze how Tier 1 issues affect the building. This could mean testing different earthquake scenarios to see how the structure holds up. The goal is to fully understand the building's earthquake resistance and identify where precaution or strengthening are needed.

The research also introduces the idea of proportional seismic force as an additional method. Based on pushover analysis principles, it gives us a look at how strong existing buildings are. Its goal is to show how buildings perform if they don't meet ASCE 41-47 standards. By applying earthquake forces gradually and studying how the building reacts, this analysis uncovers potential failure mechanisms and structural vulnerabilities.

Basically, the research combines these methods to provide a complete framework for evaluating the seismic resistance of existing buildings. By carefully studying and analyzing, it aims to inform necessary precautions or strengthening to enhance overall structural safety.

## RESULTS AND DISCUSSION

### Structural System

From the visual observations and measurements conducted on-site, it is revealed that the case study building utilizes a moment steel frame with a wooden floor structure. The structural system plan of a typical floor is shown in Figure 1.

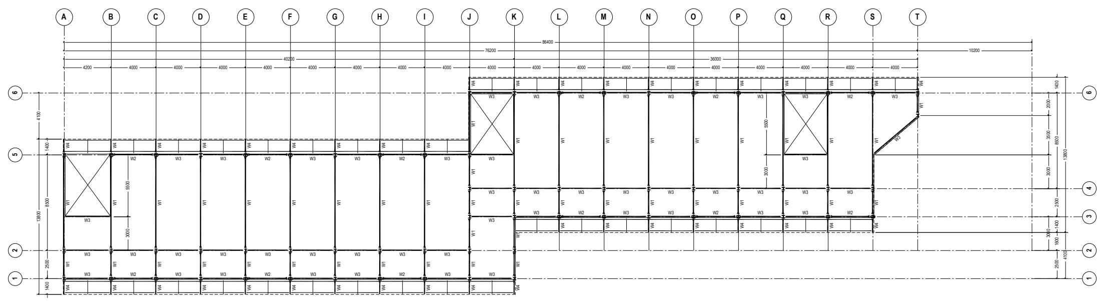


**Figure 1 . Structural System Plan of Typical Floor**

Figure 2 shows the building's structural system in the longitudinal direction, which consists of alternating moment frames connected by beams with pinned joints at both ends. The column arrangement forms an H-shape section - King Cross - King Cross pattern, repeated along the outer side of the building. Meanwhile, Figure 3 illustrates the building's structural system in the transverse direction, which consists of a series of moment frames with a gable frame on top. At each rigid frame, both ends of the beam are equipped with haunches made from the W-shape beam sections.

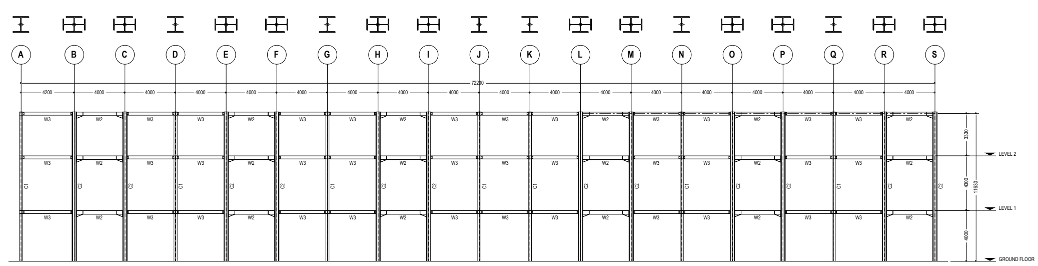


**Figure 2. Structural System Section in Longitudinal Direction**

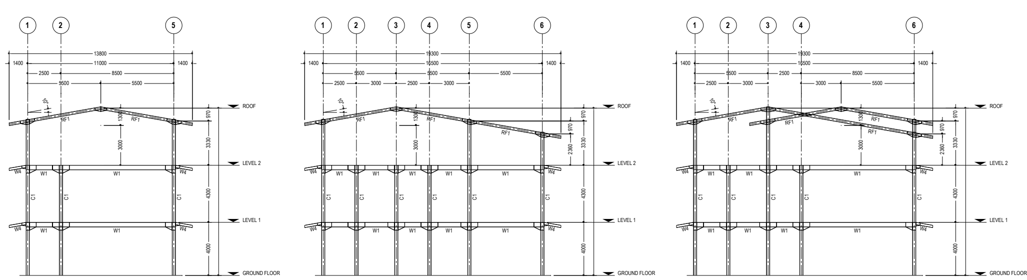


**Figure 3. Structural System Sections in Transverse Direction**

## Structural System Analysis

The mathematical model of the building's structural system, analyzed by ETABS software, is shown in Figure 4. The haunches of the beams, which are part of the lateral force-resisting system, are accurately modeled to match the existing conditions. This is intended to achieve a building behavior closer to the actual conditions. All column supports at the building base are assumed to be pinned joints in accordance with design practices in the 1980s era.

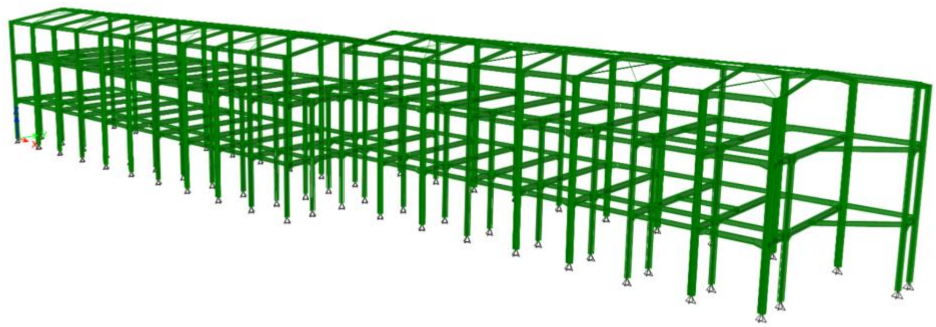
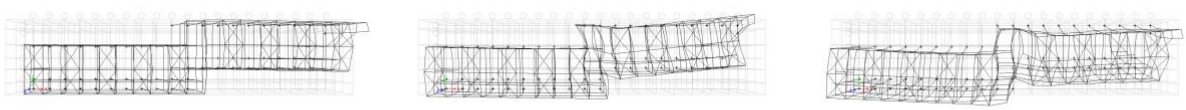


**Figure 4. ETABS Mathematical Model**

Figure 5 illustrates the case study building dynamic analysis results. The first and third modes are translational modes, while the second mode is a rotational mode. This indicates that the building's behavior is less ideal, and it is sensitive to torsion.



(a) (b) (c)

**Figure 5. Mode shapes (a) Translation X Direction; (b) Rotation; (c) Translation Y Direction**

With this mathematical model, an examination of irregularities in the building's structural system was then conducted, as shown in Table 1, according to the provisions of Table 13 and Table 14 of SNI 1726:2019, along with other relevant articles. The examination results indicate the presence of horizontal irregularities such as torsional, extreme torsional, and reentrant corner irregularities, as well as vertical irregularities such as stiffness-soft story irregularity.

The extreme torsional horizontal irregularity and the stiffness-soft story vertical irregularity indicate that the behavior of the building's structural system is less favorable due to excessive torsional tendencies and the potential for soft story hazards. Both of these should be avoided when designing the building. We need to take into account that in the 1980s, building designs generally relied on two-dimensional static analysis, and Indonesian steel regulations did not yet incorporate seismic design requirements. Therefore, torsion and soft story issues were likely not detected at that time.

**Table 1. Irregularity Condition of Structure**

| Horizontal Irregularity | | | | | |
|---|---|---|---|---|---|
| **Torsional** | **Extreme torisional** | **Reentrant corner** | **Diaphragm discontinuity** | **Out of plane offset** | **Non-paralel system** |
| Yes | Yes | Yes | No | No | No |

| Vertical Irregularity | | | | | |
|---|---|---|---|---|---|
| **Stiffness - soft story** | **Stiffness - extreme soft story** | **Weight (Mass)** | **Vertical geometry** | **In-plane discontinuity in vertical lateral force-resisting element** | **Discontinuity in lateral strength - weak story** |
| Yes | No | No | No | No | No |

## Evaluation Based On The Latest SNI for Steel Structures

Based on the provisions of SNI 1726:2019, any structure located in Jakarta will be classified into Seismic Design Category D and is required to use a special moment frame, as specified in Table 12 of SNI 1726:2019. As a consequence, beams, and columns that are part of the lateral force-resisting system shall meet the requirements outlined in Section E3.6b of SNI 7860:2020. These requirements mandate that these structural elements be classified as Highly Ductile (HD) members.

To classify beams and columns as Highly Ductile (HD) members, the $b/t_f$ and $h/t_w$ ratios shall meet two requirements: first, they shall meet the compactness criteria according to Table B4.1 of SNI 1729:2020, and second, they shall meet the Highly Ductile (HD) requirements listed in Table D1.1 of SNI 7860:2020. Both of these requirements need to be met to ensure that the elements can avoid premature fracture, which leads to low cyclic resistance, thus preventing them from developing a 4% rotation capacity, as local buckling will occur before the formation of plastic hinges.

From the information provided in Table 2, it can be seen that only the W200x100 beam meets the requirements as a compact and Highly Ductile (HD) section, while the W300x150 and W350x175 beams only meet the requirements as a compact and Moderately Ductile (MD) section.

**Table 2. Ductility Condition of Beams ($R_y$ = 1.5)**

| Section | $b/t_f$ | Requirements | | | $h/t_w$ | Requirements | | | Conclusion |
|---|---|---|---|---|---|---|---|---|---|
| | | Compact $0.38\sqrt{\frac{E}{F_y}}$ | Highly Ductile $0.32\sqrt{\frac{E}{R_y F_y}}$ | Moderately Ductile $0.40\sqrt{\frac{E}{R_y F_y}}$ | | Compact $3.76\sqrt{\frac{E}{F_y}}$ | Highly Ductile $2.27\sqrt{\frac{E}{R_y F_y}}$ | Moderately Ductile $2.59\sqrt{\frac{E}{R_y F_y}}$ | |
| W 200x100 | 6.3 | 11.0 | 7.4 | 9.2 | 33.5 | 108.5 | 52.3 | 59.8 | Compact (HD) |
| W 300x150 | 8.3 | 11.0 | 7.4 | 9.2 | 43.4 | 108.5 | 52.3 | 59.8 | Compact (MD) |
| W 350x175 | 8.0 | 11.0 | 7.4 | 9.2 | 36.9 | 108.5 | 52.3 | 59.8 | Compact (MD) |

Next, from the data in Table 3, it can be observed that all columns only meet the requirements as a compact and Moderately Ductile (MD) section.

**Table 3. Ductility Condition of Columns ($R_y$ = 1.5)**

| Section | $b/t_f$ | Requirements | | | $h/t_w$ | Requirements | | | Conclusion |
|---|---|---|---|---|---|---|---|---|---|
| | | Compact $0.56\sqrt{\frac{E}{F_y}}$ | Highly Ductile $0.32\sqrt{\frac{E}{R_y F_y}}$ | Moderately Ductile $0.40\sqrt{\frac{E}{R_y F_y}}$ | | Compact $1.49\sqrt{\frac{E}{F_y}}$ | Highly Ductile $1.57\sqrt{\frac{E}{R_y F_y}}$ | Moderately Ductile $1.57\sqrt{\frac{E}{R_y F_y}}$ | |
| W 250x250 | 8.9 | 16.2 | 7.4 | 9.2 | 247 | 43.0 | 36.3 | 36.3 | Compact (MD) |
| K 250x250 | 8.9 | 16.2 | 7.4 | 9.2 | 25.8 | 43.0 | 36.3 | 36.3 | Compact (MD) |
| K 350x175 | 8.0 | 16.2 | 7.4 | 9.2 | 23.4 | 43.0 | 36.3 | 36.3 | Compact (MD) |

The beam-column connections, which are part of the special moment frames, are required to meet the provisions outlined in Section E3.6c of SNI 7860:2020. These requirements mandate the use of prequalified connections referring to SNI 7972:2020.

Figure 6 shows that the existing beam-column connections use an end plate connection system with haunches both in the transverse and longitudinal directions of the building. These connection types cannot be classified as prequalified end plate connections according to the provisions in SNI 7972:2020.

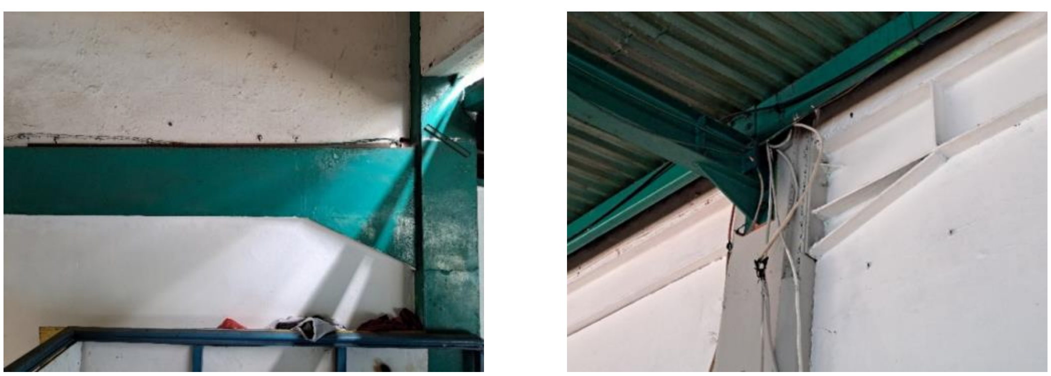


**Figure 6. Existing Condition of Beam-Column Joint Connection**

## Building Performance Level

Taking into account the age and existing condition of the building, it has been decided that the desired level of building performance to be achieved is the Limited Performance Objectives. It is the lowest level of the evaluation of the existing building in ASCE 41-17.

To achieve the Limited Performance Objectives level as per ASCE 41-17, this building needs to meet the Life Safety structural performance requirements at the BSE-1E seismic level, which refers to an earthquake with a return period of 225 years (as indicated by the blue line in Figure 7). However, the evaluation results indicate that the structural performance requirements cannot be met. Therefore, the structural performance is downgraded to Collapse Prevention but still at the BSE-1E (225-year) seismic level. The evaluation of this downgraded structural performance will be further explained in this paper.

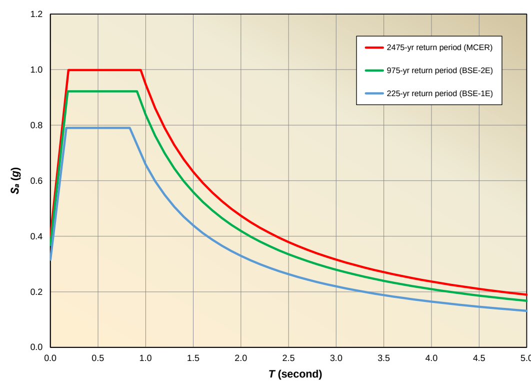



**Figure 7. Spectrum Response Graphs ($MCE_R$, BSE-2E and BSE-1E)**

## Tier 1 Evaluation

The focus of the Tier 1 Evaluation of ASCE 41-17 is on a quick assessment through visual observations on-site and available construction data. Evaluation is conducted by filling out forms that include several simple calculations, the formulas for which are provided in ASCE 41-17.

**Table 4 . Tier 1 Evaluation Result**

| Examination | Drift | Column Axial Stress | Flexural Stress | Panel Zones | Strong Column-Weak Beam | Connection |
|---|---|---|---|---|---|---|
| **Result** | 0.034 | 20 | 111 | 1,565 | 0.74 | 913 |
| **Limit** | 0.03 | 72 | 240 | 292 | 1.5 | 425 |
| **Ratio** | 1.13 | 0.28 | 0.46 | 5.36 | 2.03 | 2.15 |

Table 17-2. Collapse Prevention Basic Configuration Checklist

| Status | Evaluation Statement | Tier 2 Reference | Commentary Reference |
|---|---|---|---|
| **Low Seismicity** | | | |
| **Building System—General** | | | |
| C NC N/A U | LOAD PATH: The structure contains a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. | 5.4.1.1 | A.2.1.1 |
| C NC N/A U | ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 0.25% of the height of the shorter building in low seismicity, 0.5% in moderate seismicity, and 1.5% in high seismicity. | 5.4.1.2 | A.2.1.2 |
| C NC N/A U | MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. | 5.4.1.3 | A.2.1.3 |
| **Building System—Building Configuration** | | | |
| C NC N/A U | WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. | 5.4.2.1 | A.2.2.2 |
| C NC N/A U | SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. | 5.4.2.2 | A.2.2.3 |
| C NC N/A U | VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. | 5.4.2.3 | A.2.2.4 |
| C NC N/A U | GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. | 5.4.2.4 | A.2.2.5 |
| C NC N/A U | MASS: There is no change in effective mass of more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. | 5.4.2.5 | A.2.2.6 |
| C NC N/A U | TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. | 5.4.2.6 | A.2.2.7 |

Table 17-2 *(Continued)*. Collapse Prevention Basic Configuration Checklist

| Status | Evaluation Statement | Tier 2 Reference | Commentary Reference |
|---|---|---|---|
| **Moderate Seismicity (Complete the Following Items in Addition to the Items for Low Seismicity)** | | | |
| **Geologic Site Hazards** | | | |
| C NC N/A U | LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance do not exist in the foundation soils at depths within 50 ft (15.2 m) under the building. | 5.4.3.1 | A.6.1.1 |
| C NC N/A U | SLOPE FAILURE: The building site is located away from potential earthquake-induced slope failures or rockfalls so that it is unaffected by such failures or is capable of accommodating any predicted movements without failure. | 5.4.3.1 | A.6.1.2 |
| C NC N/A U | SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. | 5.4.3.1 | A.6.1.3 |
| **High Seismicity (Complete the Following Items in Addition to the Items for Moderate Seismicity)** | | | |
| **Foundation Configuration** | | | |
| C NC N/A U | OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. | 5.4.3.3 | A.6.2.1 |
| C NC N/A U | TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. | 5.4.3.4 | A.6.2.2 |

*Note:* C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

**Figure 8. Collapse Prevention Basic Configuration Checklist**

Table 17-8. Collapse Prevention Structural Checklist for Building Types S1 and S1a

| Status | Evaluation Statement | Tier 2 Reference | Commentary Reference |
|---|---|---|---|
| **Low Seismicity** | | | |
| **Seismic-Force-Resisting System** | | | |
| C NC N/A U | REDUNDANCY: The number of lines of moment frames in each principal direction is greater than or equal to 2. | 5.5.1.1 | A.3.1.1.1 |
| C NC N/A U | DRIFT CHECK: The drift ratio of the steel moment frames, calculated using the Quick Check procedure of Section 4.4.3.1, is less than 0.030. | 5.5.2.1.2 | A.3.1.3.1 |
| C NC N/A U | COLUMN AXIAL STRESS CHECK: The axial stress caused by gravity loads in columns subjected to overturning forces is less than $0.10F_y$. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.4.3.6, is less than $0.30F_y$. | 5.5.2.1.3 | A.3.1.3.2 |
| C NC N/A U | FLEXURAL STRESS CHECK: The average flexural stress in the moment frame columns and beams, calculated using the Quick Check procedure of Section 4.4.3.9, is less than $F_y$. Columns need not be checked if the strong column–weak beam checklist item is compliant. | 5.5.2.1.2 | A.3.1.3.3 |
| **Connections** | | | |
| C NC N/A U | TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel frames. | 5.7.2 | A.5.2.2 |
| C NC N/A U | STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation. | 5.7.3.1 | A.5.3.1 |
| **Moderate Seismicity (Complete the Following Items in Addition to the Items for Low Seismicity)** | | | |
| **Seismic-Force-Resisting System** | | | |
| C NC N/A U | REDUNDANCY: The number of bays of moment frames in each line is greater than or equal to 2. | 5.5.1.1 | A.3.1.1.1 |
| C NC N/A U | INTERFERING WALLS: All concrete and masonry infill walls placed in moment frames are isolated from structural elements. | 5.5.2.1.1 | A.3.1.2.1 |
| C NC N/A U | MOMENT-RESISTING CONNECTIONS: All moment connections can develop the strength of the adjoining members based on the specified minimum yield stress of steel. | 5.5.2.2.1 | A.3.1.3.4 |

Table 17-8 *(Continued)*. Collapse Prevention Structural Checklist for Building Types S1 and S1a

| Status | Evaluation Statement | Tier 2 Reference | Commentary Reference |
|---|---|---|---|
| **High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)** | | | |
| **Seismic-Force-Resisting System** | | | |
| C NC N/A U | MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members or panel zones based on 110% of the expected yield stress of the steel in accordance with AISC 341, Section A3.2. | 5.5.2.2.1 | A.3.1.3.4 |
| C NC N/A U | PANEL ZONES: All panel zones have the shear capacity to resist the shear demand required to develop 0.8 times the sum of the flexural strengths of the girders framing in at the face of the column. | 5.5.2.2.2 | A.3.1.3.5 |
| C NC N/A U | COLUMN SPLICES: All column splice details located in moment-resisting frames include connection of both flanges and the web. | 5.5.2.2.3 | A.3.1.3.6 |
| C NC N/A U | STRONG COLUMN—WEAK BEAM: The percentage of strong column–weak beam joints in each story of each line of moment frames is greater than 50%. | 5.5.2.1.5 | A.3.1.3.7 |
| C NC N/A U | COMPACT MEMBERS: All frame elements meet section requirements in accordance with AISC 341, Table D1.1, for moderately ductile members. | 5.5.2.2.4 | A.3.1.3.8 |
| **Diaphragms (Stiff or Flexible)** | | | |
| C NC N/A U | OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the moment frames extend less than 25% of the total frame length. | 5.6.1.3 | A.4.1.5 |
| **Flexible Diaphragms** | | | |
| C NC N/A U | CROSS TIES: There are continuous cross ties between diaphragm chords. | 5.6.1.2 | A.4.1.2 |
| C NC N/A U | STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. | 5.6.2 | A.4.2.1 |
| C NC N/A U | SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing. | 5.6.2 | A.4.2.2 |
| C NC N/A U | DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1. | 5.6.2 | A.4.2.3 |
| C NC N/A U | OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing. | 5.6.5 | A.4.7.1 |

*Note:* C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

**Figure 9 . Collapse Prevention Structural Checklist**

As shown in Figures 8 and 9, Tier 1 Evaluation identifies 9 types of deficiencies: torsion, drift check, diagonally sheathed and unblocked diaphragms, interfering walls, soft story, ties between foundation elements, strong column-weak beam, moment-resisting connections, and panel zones. All identified deficiencies must be re-evaluated using Tier 2 by applying all the consequences required by ASCE 41-17 for each deficiency.

**Tier 2 Evaluation**

Tier 2 Evaluation of ASCE 41-17 provides a more specific analysis compared to Tier 1. According to the provisions in ASCE 41-17 for Tier 2 Evaluation, only the deficiencies from Tier 1 Evaluation needed to be evaluated. A representative analysis model is required to conduct Tier 2 Evaluation, taking into account all the consequences of deficiencies from Tier 1 Evaluation to obtain the forces acting on the structural elements during a BSE-1 seismic level (225-year return period). Subsequently, the forces acting are compared with the amplified structural element capacities as per the provisions of ASCE 41-17.

As shown in Table 5, beam-column connection is the only item that still indicates failure. Thus, the remaining deficiency after conducting Tier 2 Evaluation is the beam-column connection and inter-story drift due to connection failure.

If referring to the steps in ASCE 41-17, all deficiencies at Tier 2 need to be re-evaluated with Tier 3 using nonlinear analysis. However, nonlinear analysis does not accommodate connection checks. Therefore, the evaluation is stopped at Tier 2 of ASCE 41-17.

**Table 5. Tier 2 Evaluation Result**

| Examination | Beam | Column | Connection | Panel Zone |
|---|---|---|---|---|
| ***m*** | 3.57 | 3.1 | 3 | 10 |
| **Calculation** | 576 | 0.63 | 1,771 | 868 |
| **Limit** | 694 | 1 | 1,274 | 3,264 |
| **Ratio** | 0.83 | 0.63 | 1.39 | 0.27 |

**Proportional Seismic Force Method**

After evaluating the latest building standards in Indonesia (SNI) and ASCE 41-17, it is apparent that the structure does not meet the permissible acceptance criteria, especially regarding connections and structural deformations. With the understanding that significant deformations can be acceptable if the structural element's strength can accommodate them, inspired by the pushover analysis, the concept of proportional seismic force is introduced to assess the actual performance of buildings that cannot meet the acceptance criteria of both new and existing building regulations.

The requirement that shall be maintained in the concept of proportional seismic force is the ductility of the elements. These elements shall meet the ductility provisions specified in the latest building design regulations. However, based on the analysis results listed in Table 2 and Table 3, the structural elements of this building do not meet these ductility requirements. It is important to note that the $R_y$ value used in these checks was 1.5, as required by the latest building regulations. However, it should be remembered that this building was not designed in this decade but rather in the 1980s when steel production specifications differed from current standards. Therefore, it is considered to use the $R_y$ value specified in ASCE 41-17 for steel materials produced in the period from 1961 to 1990, which is 1.1.

As shown in Table 6 and Table 7, almost all elements have met the ductility requirements, with only the column elements failing to meet the ductility requirements by just 1%. Based on these results, it is believed that further analysis and evaluation using the proportional seismic force concept are still feasible.

**Table 6. Ductility Condition of Beams ($R_y$ = 1.1)**

| Section | $b/t_f$ | Requirements: Compact $0.38\sqrt{\frac{E}{F_y}}$ | Requirements: Highly Ductile $0.32\sqrt{\frac{E}{R_y F_y}}$ | Requirements: Moderately Ductile $0.40\sqrt{\frac{E}{R_y F_y}}$ | $h/t_w$ | Requirements: Compact $3.76\sqrt{\frac{E}{F_y}}$ | Requirements: Highly Ductile $2.27\sqrt{\frac{E}{R_y F_y}}$ | Requirements: Moderately Ductile $2.59\sqrt{\frac{E}{R_y F_y}}$ | Conclusion |
|---|---|---|---|---|---|---|---|---|---|
| W 200x100 | 6.3 | 11 | 8.8 | 11.0 | 33.5 | 108.5 | 62.4 | 71.2 | Compact (HD) |
| W 300x150 | 8.3 | 11 | 8.8 | 11.0 | 43.4 | 108.5 | 62.4 | 71.2 | Compact (HD) |
| W 350x175 | 8.0 | 11 | 8.8 | 11.0 | 36.9 | 108.5 | 62.4 | 71.2 | Compact (HD) |

**Table 7. Ductility Condition of Columns ($R_y$ = 1.1)**

| Section | $b/t_f$ | Requirements: Compact $0.56\sqrt{\frac{E}{F_y}}$ | Requirements: Highly Ductile $0.32\sqrt{\frac{E}{R_y F_y}}$ | Requirements: Moderately Ductile $0.40\sqrt{\frac{E}{R_y F_y}}$ | $h/t_w$ | Requirements: Compact $1.49\sqrt{\frac{E}{F_y}}$ | Requirements: Highly Ductile $1.57\sqrt{\frac{E}{R_y F_y}}$ | Requirements: Moderately Ductile $1.57\sqrt{\frac{E}{R_y F_y}}$ | Conclusion |
|---|---|---|---|---|---|---|---|---|---|
| W 250x250 | 8.9 | 16.2 | 8.8 | 11.0 | 24.67 | 43.0 | 43.2 | 43.2 | Compact (MD) |
| K 250x250 | 8.9 | 16.2 | 8.8 | 11.0 | 25.83 | 43.0 | 43.2 | 43.2 | Compact (MD) |
| K 350x175 | 8.0 | 16.2 | 8.8 | 11.0 | 23.43 | 43.0 | 43.2 | 43.2 | Compact (HD) |

After a series of extensive trials, by adjusting the seismic force reduction factor, a reduction factor was obtained that did not result in failure at the connections and was able to meet the requirements for inelastic deformation of the structure, which is 12.5, as shown in Table 8. Although not included in this paper, it is actually possible to obtain the earthquake return period value that the building can withstand.

**Table 8. Comparison of Results between SNI 1726:2019 and Proportional Seismic Forces**

| Design Concept | Seismic Reduction Factor | D/C Ratio: Drift | D/C Ratio: Beam | D/C Ratio: Column | D/C Ratio: Connection |
|---|---|---|---|---|---|
| SNI 1726:2019 | 8 | 1.47 | 0.87 | 0.79 | 2.96 |
| Proportional Seismic Forces | 12.5 | 0.94 | 0.53 | 0.57 | 1.00 |

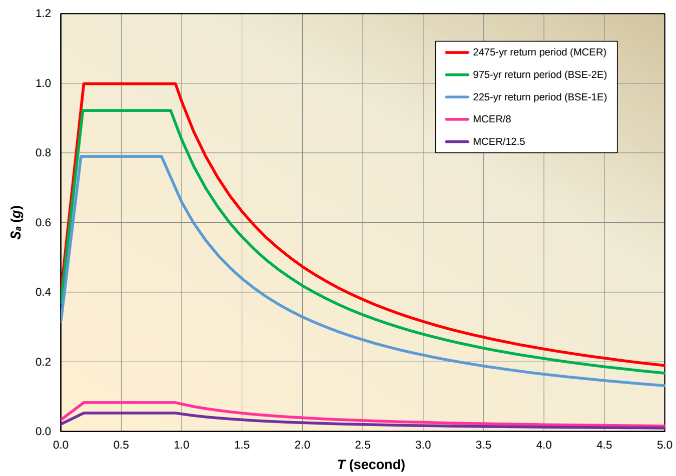



**Figure 10. Comparison of Seismic Levels**

## CONCLUSION

The evaluation results reveal critical issues with this building. The structure has a significant risk of excessive torsion and a highly dangerous potential soft story at Level 1. Additionally, the beams and columns fail to meet the seismic provisions and detailing requirements of SNI 7860:2020 and SNI 7972:2020, resulting in low cyclic resistance and inability to achieve a 4% rotation capacity due to local buckling occurring before plastic hinge formation. The building also does not meet the lowest performance level of acceptance criteria in the ASCE 41-17 standards. Furthermore, the building can only withstand earthquakes up to MCER/12.5, with an estimated first failure at the beam-column connections. Based on these findings, there are two recommendations: either retain the building for no more than five years with facade improvements using comparable or lighter materials, or demolish the building and construct a new one that meets functional requirements and complies with all the latest building codes.

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