# Structural evaluation and analysis of existing house renovation into shophouse based on Building Information Modeling

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**Abstract**

*Indonesia's population and economic growth must be supported by adequate facilities and infrastructure, including residences and business premises. Renovating residential houses into shophouses offers an effective way to optimize land use and reduce construction costs. A thorough evaluation of existing structural elements is critical to determining which components can be retained, alongside accurate cost and scheduling assessments to ensure smooth renovation. This research utilizes Building Information Modeling (BIM) to integrate structural, architectural, and MEP work, as well as to streamline scheduling and cost estimation. ETABS is employed for structural design, while Autodesk Revit and Navisworks are used for cost and schedule planning. The assessment follows ASCE 41-17 guidelines to evaluate existing structures, and SNI 1726:2019 and SNI 2847:2019 standards for retrofitting and designing new structures. The evaluation reveals deficiencies in global strength and component ductility, necessitating structural retrofits using concrete jacketing. This study not only addresses the renovation of a residential house into a shophouse but also contributes to broader research in construction. The estimated cost for the structural renovation is Rp. 159,692,607.93, and the duration is projected at 58 days, with 11 days for demolition and 47 days for upper structure construction.*



## Introduction

The global economy faced a significant slowdown due to the COVID-19 pandemic; however, Indonesia's economic growth has continued to rise steadily. According to data from the Central Statistics Agency, Indonesia’s economic growth was 3.69% in 2020, increasing to 5.02% in 2021, and reaching 5.31% in 2022 (Statistics Indonesia, 2021, 2022). Alongside this economic growth, Indonesia has also seen a rise in population. In 2020, 56.7% of the population lived in urban areas, and this figure is projected to increase to 66.6% by 2035 (Statistics Indonesia, 2020). However, this rapid population growth is not matched by the availability of adequate infrastructure, including housing and commercial spaces. Without proper planning, this imbalance could lead to overcrowding and a decline in the quality of urban life. Addressing this issue requires innovative solutions that optimize land use and improve infrastructure development.

People living in urban areas often seek to start businesses to increase their income. However, the lack of available land forces them to find alternative solutions, one of which is renovating residential houses into shophouses. Using a residence as a place of business offers greater efficiency and mobility for the user. Additionally, residential houses tend to undergo changes and developments from their original design, making renovation necessary over time.

Renovating a residential house into a shophouse is often chosen to reduce land use and minimize construction costs. This approach is effective if the renovation is done properly and the existing structure meets the requirements of the additional load. However, it becomes impractical if the existing structure is in poor condition and cannot support the new design. To ensure effective and efficient renovations, structural optimization is necessary, aligning the condition of the existing structure with the specifications of the planned changes. The structural design must meet safety and strength standards, accounting for the combination of loads acting on the building. Additionally, assessing the existing structural elements is crucial to determine their capacity to bear the applied loads, which helps identify elements that can be maintained, need modification, or must be replaced.

It is also important to know the costs and duration of the renovation to ensure the project runs smoothly. One technology that aids in visualizing, scheduling, and displaying the required volume for a construction project is Building Information Modeling (BIM). BIM enables accurate cost estimation and time performance assessments before the work begins, helping to streamline the renovation process.

Building Information Modeling (BIM) supports the implementation of structural, architectural, and MEP work, as well as scheduling and cost estimation in construction projects. BIM tools work seamlessly together; for example, Autodesk Revit is used to visualize the 3D design of buildings and can also estimate volume and cost. The 3D model from Autodesk Revit can then be reviewed in Autodesk Navisworks, allowing for scheduling to be developed and simulated, showing the construction activities according to the project's timeline.

## Research methods

This research focuses on structural planning, cost estimation, and scheduling for renovating a residential house into a shophouse. The study uses a descriptive quantitative approach. The modeling of the existing residential structure was created using ETABS and adjusted according to the Detailed Engineering Design (DED) previously developed in AutoCAD, with material specifications based on the 2022 earthquake-resistant building construction manual issued by the Direktorat Pengembangan Kawasan Pemukiman Kementerian PUPR. The existing structure was first evaluated by following the stages outlined in ASCE 41-17, Seismic Evaluation and Retrofit of Existing Buildings. The renovation involves converting a 1-story residential house into a 3-story shophouse by adding two new floors. Structural elements include both existing and new components, designed in accordance with SNI 1726:2019 and SNI 2847:2019. Since the original 1-story structure was not designed to support the additional load from the new floors, a review and reinforcement of the existing elements are necessary, while the new elements will be designed to meet current requirements.

The completed structure was modeled in 3D using Autodesk Revit. In Revit, clash detection was performed, and cost estimation was calculated. To develop the construction schedule, Autodesk Navisworks was used to generate a scheduling simulation. The research conceptual framework is shown in Figure 1.

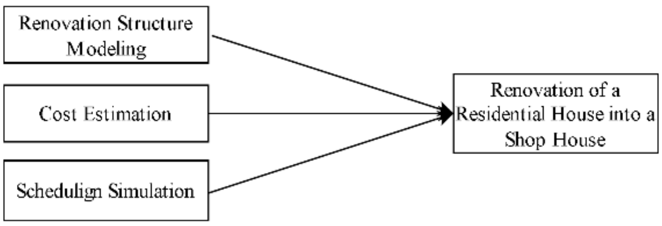


Figure 1. Research conceptual framework

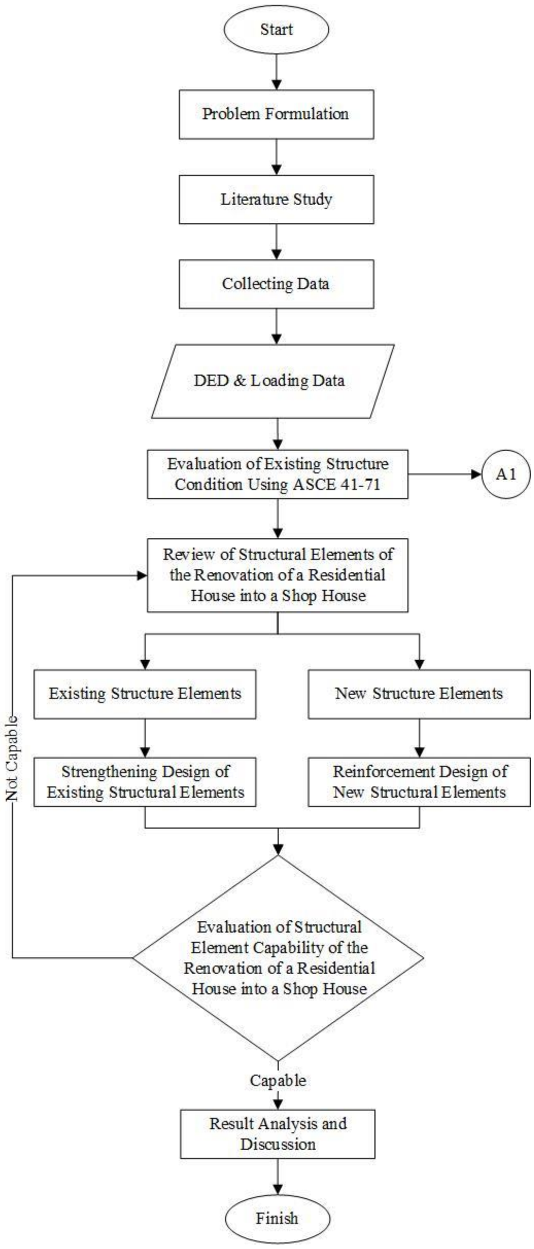


Figure 2. The research flow

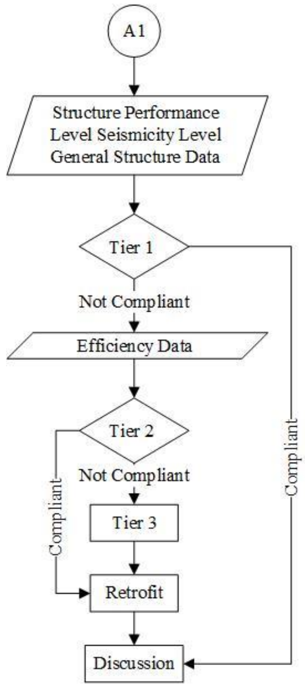


Figure 3. Existing structure evaluation flow

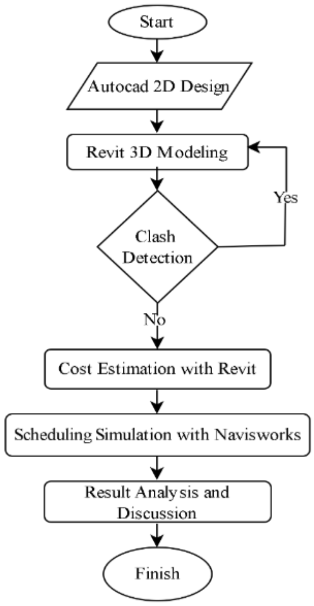


Figure 4. BIM method flow

## Results and discussion

### *Evaluation of existing structure condition*

The existing structure is first evaluated by following the stages outlined in ASCE 41-17, which include Tier 1 and Tier 2 assessments, to determine whether strengthening is required. According to ASCE 41-17, for seismic rehabilitation of existing buildings, two levels of seismicity are considered: BSE-1E and BSE-2E earthquakes. The basic performance objective for buildings in risk categories I and II is life safety under BSE-1E earthquake conditions, and collapse prevention under BSE-2E earthquake conditions. However, the use of BSE-1E for Tier 1 and Tier 2 evaluations in buildings within risk categories I-III is not required. Based on research by Rashid (2021), structural evaluation after the addition of levels produced a ratio value greater than 1, indicating that the structure is in an over-stress (O/S) state. A comparison of the spectral response curves for MCE, BSE-1N, and BSE-2E earthquakes is shown in Figure 5. The calculations for converting MCE earthquake levels to BSE-2E levels are as follows:

$$S_{S(975)} = S_{S(2475)} \times \left(\frac{975}{2475}\right)^{k}$$

$$= 0{,}8366 \times \left(\frac{975}{2475}\right)^{0{,}35}$$

$$= 0{,}6038 \text{ g}$$

$$S_{1(975)} = S_{1(2475)} \times \left(\frac{975}{2475}\right)^{k}$$

$$= 0{,}4064 \times \left(\frac{975}{2475}\right)^{0{,}35}$$

$$= 0{,}2933 \text{ g}$$

$$S_{XS(975)} = F_a \times S_{S(975)}$$

$$= 1{,}3169 \times 0{,}6038$$

$$= 0{,}7952 \text{ g}$$

$$S_{X1(975)} = F_v \times S_{1(975)}$$

$$= 2{,}0133 \times 0{,}2933$$

$$= 0{,}5906 \text{ g}$$

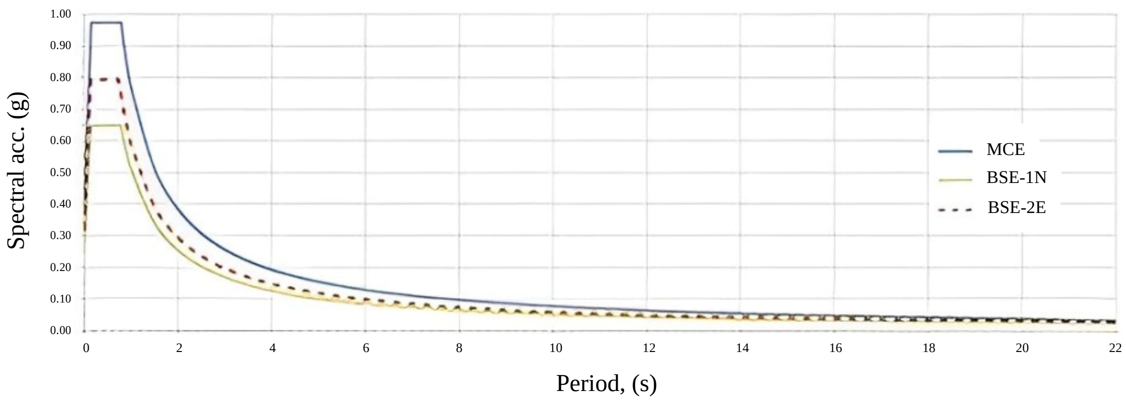


Figure 5. Comparison of spectrum response curves

### *Tier 1 Analysis*

Tier 1 analysis involves a basic configuration checklist and a structural checklist tailored to the specific building type. It requires simple calculations to assess the existing structure based on the criteria outlined in ASCE 41-17. The details of the collapse prevention basic configuration checklist are provided in Table 1, and the structural collapse prevention checklist is shown in Table 2.

Table 1. Collapse prevention basic configuration checklist

| Status | | | | Evaluation Statement |
|---|---|---|---|---|
| Low Seismicity | | | | |
| Building System (General) | | | | |
| C | NC | N/A | U | Load Path |
| C | NC | N/A | U | Adjacent Buildings |
| C | NC | N/A | U | Mezzanines |
| Building System (Building Configuration) | | | | |
| C | NC | N/A | U | *Weak Story* |
| C | NC | N/A | U | *Soft Story* |

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| Status | | | | Evaluation Statement |
|---|---|---|---|---|
| C | NC | N/A | U | Vertical Irregularities |

| Status | | | | Evaluation Statement |
|---|---|---|---|---|
| Building System (Building Configuration) | | | | |
| C | NC | N/A | U | Geometry |
| C | NC | N/A | U | Mass |
| C | NC | N/A | U | Torsion |
| Moderate Seismicity | | | | |
| Geologic Site Hazards | | | | |
| C | NC | N/A | U | Liquefaction |
| C | NC | N/A | U | Slope Failure |
| C | NC | N/A | U | Surface Fault Rupture |
| High Seismicity | | | | |
| Foundation Configuration | | | | |
| C | NC | N/A | U | *Overturning* |
| C | NC | N/A | U | *Ties Between Foundation* |

| Status | | | | Evaluation Statement |
|---|---|---|---|---|
| Low Seismicity | | | | |
| Seismic-Force-Resisting System | | | | |
| C | NC | N/A | U | Redundancy |
| C | NC | N/A | U | Column Axial Stress Check |
| Connections | | | | |
| C | NC | N/A | U | Concrete Columns |
| Moderate Seismicity | | | | |
| Seismic-Force-Resisting System | | | | |
| C | NC | N/A | U | Redundancy |
| C | NC | N/A | U | *Interfering Walls* |
| C | NC | N/A | U | Column Shear Stress Check |
| C | NC | N/A | U | *Flat Slab Frames* |
| High Seismicity | | | | |
| Seismic-Force-Resisting System | | | | |
| C | NC | N/A | U | Prestressed Frame Elements |
| C | NC | N/A | U | *Captive Columns* |
| C | NC | N/A | U | No Shear Failure |
| C | NC | N/A | U | Strong Column-Weak Beam |

| Status | | | | Evaluation Statement |
|---|---|---|---|---|
| High Seismicity | | | | |
| Seismic-Force-Resisting System | | | | |
| C | NC | N/A | U | Beam Bars |
| C | NC | N/A | U | Column-Bar Splices |
| C | NC | N/A | U | Beam-Bar Splices |
| C | NC | N/A | U | Column-Tie Spacing |
| C | NC | N/A | U | Stirrup Spacing |
| C | NC | N/A | U | Joint Transverse Reinforcing |
| C | NC | N/A | U | Deflection Compatibility |
| C | NC | N/A | U | Flat Slabs |
| Diaphragms | | | | |
| C | NC | N/A | U | Diaphragm Continuity |
| Connections | | | | |
| C | NC | N/A | U | Uplift at Pile Caps |

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

a. Period (T)

The fundamental period of the structure can be calculated using the equation:

$$T = C_t\, h_n^{\beta} \qquad (1)$$

Note: $C_t$ = 0,018 for moment-resisting frames of reinforced concrete, $h_n$ = height above the base to the roof level (ft), $\beta$ = 0,90 for moment-resisting frames of reinforced concrete.

$$T = 0{,}018 \times 10{,}499^{0{,}9}$$

$$= 0{,}149 \text{ s}$$

b. Spectral acceleration ($S_a$)

Spectral acceleration values are derived from the BSE-2E earthquake response spectrum. The obtained spectral acceleration value is 0.793 g.

c. Pseudo seismic force (V)

The pseudo seismic force can be calculated using the equation:

$$V = C\, S_a\, W \qquad (2)$$

Note: C = Modification factor (Taken from table 4-7 ASCE 41-17), $S_a$ = spectral acceleration, W = effective seismic weight of the structure.

V $= 1{,}3 \times 0{,}793 \times 78{,}291$

$= 80{,}744$ kN

d. Shear stress in concrete frame columns ($V_j^{avg}$)

The average shear stress in concrete frame columns can be calculated using the equation:

$$V_j^{avg} = \frac{1}{M_s}\left(\frac{n_c}{n_c - n_f}\right)\frac{V_j}{A_c} \quad (3)$$

Note: $M_s$ = System modification factor (2.0 for buildings being evaluated to the collapse prevention performance level), $n_c$ = total number of columns, $n_f$ = total numbers of frames in the direction of loading, $V_j$ = story shear, $A_c$ = summation of the cross-sectional area of all columns in the story under consideration.

$$V_j^{avg} = \frac{1}{2} \times \left(\frac{14}{14-2}\right) \times \frac{80{,}744}{0{,}315}$$

$= 0{,}150$ MPa

To meet the requirements stated in ASCE 41-17, the $V_j^{avg}$ value must less than the largest value between 100 lb/in² (0.69 MPa) or $0.17\sqrt{fc'}$ so the structure under review meets the requirements.

e. Column axial stress caused by overturning ($p_{ot}$)

The axial stress of columns in moment frames at the base, subjected to overturning forces, can be calculated using the following equation:

$$p_{ot} = \frac{1}{M_s}\left(\frac{2}{3}\right)\left(\frac{Vh_n}{Lnf}\right)\left(\frac{1}{A_{col}}\right) \quad (4)$$

Note: $M_s$ = System modification factor (2.5 for buildings being evaluated to the collapse prevention performance level), V = pseudo seismic force, $h_n$ = height above the base to the roof level, L = Total length of the frame, $n_f$ = total numbers of frames in the direction of loading, $A_{col}$ = area of the end column of the frame.

Y direction

$$p_{ot} = \frac{1}{2{,}5}\left(\frac{2}{3}\right)\left(\frac{80{,}744 \times 3{,}2}{6 \times 2}\right)\left(\frac{1}{0{,}0225}\right)$$

$= 0{,}255$ MPa

X direction

$$p_{ot} = \frac{1}{2{,}5}\left(\frac{2}{3}\right)\left(\frac{80{,}744 \times 3{,}2}{6{,}45 \times 2}\right)\left(\frac{1}{0{,}0225}\right)$$

$= 0{,}237$ MPa

To meet the requirements stated in ASCE 41-17, the $p_{ot}$ value must less than 0.3fc' so the structure under review meets the requirements.

f. Strong column-weak beam

ASCE 41-17 requires the sum of the columns capacity moment is 20% greater than the beams capacity moment value.

$Mn_b = 4{,}850$ kNm

$Mn_c = 7{,}884$ kNm

$\Sigma Mn_c \geq 1{,}2\ \Sigma Mn_b$

$7{,}884 \geq 1{,}2\ (4{,}850 + 4{,}850)$

$7{,}884 \leq 11{,}640$

g. Column-tie spacing

ASCE 41-17 requires that columns have ties spaced at or less than d/4 throughout the length and at or less than $8d_b$ at all locations with the potential for plastic hinge.

d/4 = 30,5 mm

$8d_b$ = 80 mm

The existing column-tie spacing is 150 mm so the requirements are not met.

h. Stirrup spacing

ASCE 41-17 requires that beams have stirrup spaced at or less than d/2 throughout the length and at or less than the smallest value between $8d_b$ or d/4.

d/2 = 61 mm

$8d_b$ = 80 mm

d/4 = 30,5 mm

The existing beams-stirrup spacing is 150 mm so the requirements are not met.

***Tier 2 Analysis***

The procedure used for the Tier 2 analysis in this study is the dynamic linear procedure. The load combinations for calculating deformation-controlled actions and force-controlled actions are as follows:

$$Q_{UD} = Q_G + Q_E \quad (5)$$

$$Q_{UF} = Q_G \pm \frac{\chi Q_E}{C_1 C_2 J} \quad (6)$$

$$Q_G = 1{,}1\ (Q_D + Q_L + Q_S) \quad (7)$$

$$Q_G = 0{,}9\ Q_D \quad (8)$$

$Q_{UD}$ = Deformation-controlled action, $Q_{UF}$ = force-controlled action, $Q_G$ = action caused by gravity loads, $Q_E$ = action caused by the response to the selected seismic hazard level, $Q_D$ = action caused by dead loads, $Q_L$ = action caused by live load, $C_1$ $C_2$ = modification factor, J = force-delivery reduction factor (2.0 for a high level of seismicity)

Deformation-controlled and force-controlled actions must meet the acceptance criteria for the linear procedure as follows:

$$m\kappa Q_{CE} > Q_{UD} \quad (9)$$

$$\kappa Q_{CL} > Q_{UF} \quad (10)$$

Note: m = Component modification factor, $Q_{CE}$ = expected strength, $Q_{CL}$ = lower-bound strength, κ = knowledge factor.

a. Column check

   $\kappa = 0{,}75$

   1. Deformation-controlled action

      $m = 1{,}357$

      $m\kappa > \frac{Q_{UD}}{Q_{CE}}$

      $1{,}018 > \frac{Q_{UD}}{Q_{CE}}$

      According to the acceptance criteria, the column capacity ratio of existing structures should not exceed 1.018. However, after evaluating the column capacity of the existing structure using ETABS, the ratio exceeded this permitted limit.

   2. Force-controlled action

      $V_u = 3{,}453$ kN

      $V_n = 37{,}123$ kN

      $\kappa > \frac{Q_{UF}}{Q_{CL}}$

      $0{,}75 > 0{,}093$

      According to the acceptance criteria, the column's nominal shear strength should be sufficient to withstand the applied shear forces.

      $P_n = 350{,}373$ kN

      $P_u = 21{,}377$ kN

      $\kappa > \frac{Q_{UF}}{Q_{CL}}$

      $0{,}75 > 0{,}061$

      According to the acceptance criteria, the column's nominal compressive strength should be adequate to withstand the applied axial forces.

b. Beam check

   $\kappa = 0{,}75$

   1. Deformation-controlled action

      $M_{UD} = 17{,}277$ kN

      $M_{nCE}^{+} = 6{,}170$ kN

      $M_{nCE}^{-} = 6{,}170$ kN

      $m = 4$

      $m\kappa > \frac{Q_{UD}}{Q_{CE}}$

      $3 > 2{,}800$

      According to the acceptance criteria, the beam capacity ratio is within the permitted limits.

   2. Force-controlled action

      $V_u = 16{,}207$ kN

      $V_n = 33{,}347$ kN

      $\kappa > \frac{Q_{UF}}{Q_{CL}}$

      $0{,}75 > 0{,}486$

      According to the acceptance criteria, the beam's nominal shear strength is sufficient to withstand the applied shear forces.

c. Joint check

$\kappa$ = 0,75

$V_j$ = 31,973 kN

$V_u$ = 63,709 kN

$$\kappa > \frac{Q_{UF}}{Q_{CL}}$$

0,75 < 1,993

According to the acceptance criteria, the joint's nominal shear strength is insufficient to withstand the applied shear forces.

### *Existing structure retrofit efforts*

Based on the Tier 1 and Tier 2 evaluations using ASCE 41-17, deficiencies related to global strength and component ductility were identified, necessitating retrofit efforts on the existing structure. Existing structures undergoing functional changes, and the addition of new components must be analyzed for their ability to support the additional loads by checking the capacity ratio and deflection. The capacity ratio check for the new structure is shown in Figure 6.

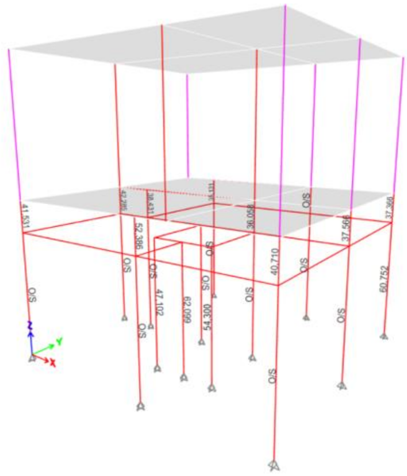

Figure 6. Capacity ratio check

The existing structure is considered capable of withstanding additional loads if the capacity ratio is less than 1 and the deviations do not exceed the limits specified by SNI 1726:2019. However, the deviation values at level 1 in both the x-direction and y-direction exceed the allowable limits, as detailed in Tables 3 and 4.

Table 2. Deflection check in x direction

| Story | h (mm) | δe (mm) | δ (mm) | Δ (mm) | Δa (mm) | Notes |
|---|---|---|---|---|---|---|
| 2 | 3500 | 450,005 | 450,005 | 17,11 | 53,8 | Within limit |
| 1 (extension) | 800 | 432,895 | 432,895 | 21,137 | 12,3 | Exceed limit |
| 1 | 3200 | 411,758 | 411,758 | 411,758 | 49,2 | Exceed limit |

Table 3. Deflection check in y direction

| Story | h (mm) | δe (mm) | δ (mm) | Δ (mm) | Δa (mm) | Notes |
|---|---|---|---|---|---|---|
| 2 | 3500 | 479,579 | 479,579 | 19,6 | 53,8 | Within limit |
| 1 (extension) | 800 | 459,979 | 459,979 | 23,248 | 12,3 | Exceed limit |
| 1 | 3200 | 436,731 | 436,731 | 436,731 | 49,2 | Exceed limit |

### *Retrofit and reinforcement design*

The retrofitting method employed is concrete jacketing. The concrete used has a strength of 22.391 MPa, and the steel used has a strength of 420 MPa.

a. Slab reinforcement design

The long-term deflection value is within the permitted limits specified by SNI 2847:2019, indicating that the floor slab design is safe. The recapitulation of reinforcement requirements is provided in Table 5.

Table 4. Slabs reinforcement detail

| Slab Section | Thickness (mm) | X Direction | | Y Direction | |
|---|---|---|---|---|---|
| | | End | Mid-span | End | Mid-span |
| Story 2 | | | | | |
| S2 3000.4250 | 120 | D10-120 | D10-240 | D10-240 | D10-240 |
| S2 3000.2200 | | D10-240 | D10-240 | D10-240 | D10-240 |
| S2 1200.2200 | | D10-240 | D10-240 | D10-240 | D10-240 |
| S2 1800.1050 | | D10-240 | D10-240 | D10-240 | D10-240 |

| Slab Section | Thickness (mm) | X Direction | | Y Direction | |
|---|---|---|---|---|---|
| | | End | Mid-span | End | Mid-span |
| Rooftop | | | | | |
| S3 3000.4250 | 100 | D10-200 | D10-200 | D10-200 | D10-200 |
| S3 3000.2200 | | D10-200 | D10-200 | D10-200 | D10-200 |
| S3 1800.2200 | | D10-200 | D10-200 | D10-200 | D10-200 |

b. Beams reinforcement design

The long-term deflection value is within the permitted limits set by SNI 2847:2019, confirming that the floor slab design is safe. The recapitulation of beam bending reinforcement requirements is provided in Table 6, while the recapitulation of beam shear reinforcement requirements is shown in Table 7.

Table 5. Beams flexural reinforcement detail

| Beam Section | Length (mm) | End Beam Reinforcement | | Mid-span Beam Reinforcement | |
|---|---|---|---|---|---|
| | | Top | Bottom | Top | Bottom |
| B2 200.250 | 2200 | 2D13 | 2D13 | 2D13 | 2D13 |
| | 3000 | 3D13 | 2D13 | 2D13 | 2D13 |
| | 4250 (A) | 4D13 | 2D13 | 2D13 | 2D13 |
| | 4250 (B) | 3D13 | 2D13 | 2D13 | 2D13 |
| B3 200.250 | 2220 | 2D13 | 2D13 | 2D13 | 2D13 |
| | 3000 | 2D13 | 2D13 | 2D13 | 2D13 |
| | 4250 | 3D13 | 2D13 | 2D13 | 2D13 |
| BA 150.200 | - | 2D13 | 2D13 | 2D13 | 2D13 |

Table 6. Beams transverse reinforcement detail

| Beam Section | Transverse Reinforcement | |
|---|---|---|
| | End beam | Mid-span beam |
| B2 200.250 | D8-45 | D8-90 |
| B3 200.250 | D8-45 | D8-90 |
| BA 150.200 | D8-35 | D8-75 |

c. Columns reinforcement and retrofit design

The recapitulation of column reinforcement requirements is listed in Table 8.

Table 7. Columns reinforcement detail

| Column Section | Length (mm) | Flexural Reinforc. | Mid-span Beam Reinforcement | |
|---|---|---|---|---|
| | | | Top | Bottom |
| KJ 300.300 | 3200 | 4D19 | D10-75 | D10-100 |
| K1 300.300 | 4000 | 4D19 | D10-75 | D10-100 |
| K1 300.300 | 800 | 4D19 | D13-75 | D13-75 |
| K2 300.300 | 3500 | 4D19 | D10-75 | D10-100 |

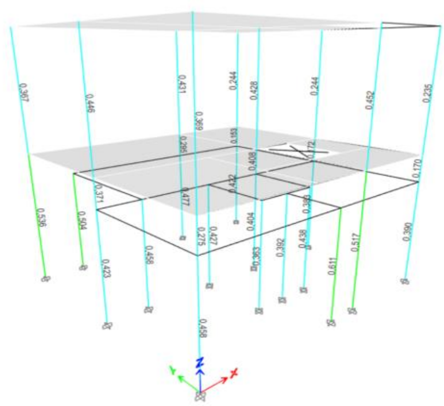

Figure 7. Capacity ratios check after retrofitting

Figure 7 shows the capacity ratio of the structure after the change in function. It can be concluded that the structural design, following the change in function and addition of levels, is safe after undergoing reinforcement. The deviation values for Story 1 and Story 2 in both the x-direction and y-direction are within the permitted limits, as shown in Tables 9 and 10.

Table 8. Deflection check in x direction after retrofitting

| Story | h (mm) | δe (mm) | δ (mm) | Δ (mm) | Δa (mm) | Notes |
|---|---|---|---|---|---|---|
| 2 | 3500 | 12,801 | 70,406 | 34,832 | 53,846 | Within limit |
| 1(extension) | 800 | 6,468 | 35,574 | 8,157 | 12,308 | Within limit |
| 1 | 3200 | 4,985 | 27,418 | 37,418 | 49,231 | Within limit |

Table 9. Deflection check in y direction after retrofitting

| Story | h (mm) | δe (mm) | δ (mm) | Δ (mm) | Δa (mm) | Notes |
|---|---|---|---|---|---|---|
| 2 | 3500 | 14,969 | 82,330 | 40,431 | 53,846 | Within limit |
| 1(extension) | 800 | 7,618 | 41,899 | 9,790 | 12,308 | Within limit |
| 1 | 3200 | 5,838 | 32,109 | 32,109 | 49,231 | Within limit |

***3D Revit modelling***

After designing the renovation structure, the 3D model is implemented by inputting the 2D images into Revit. This research divides the modeling into two parts: the existing house structure model and the shophouse structure model. The 3D models are shown in the following images.

Figure 8. Existing House 3D Model

Figure 9. Shophouse 3D Model

***Cost estimation for demolition work***

The cost estimation results are obtained by multiplying the unit price with the take-off quantity data generated by Autodesk Revit.

a. Cost estimation for wall demolition
   The calculation indicates that the cost of the wall demolition work was IDR 10,116,811.17, representing 6.335% of the total project cost.
b. Cost estimation for roof demolition
   The calculation shows that the cost of the work was IDR 6,366,994.45, accounting for 3.987% of the total project cost.
c. Cost estimation for column concrete cover demolition
   The calculation shows that the cost of the work was IDR 10,700.62, which represents 0.007% of the total project cost.

Thus, the total weight percentage for demolition work is 10.33% of the entire project, with a total cost of IDR 16,494,506.24.

***Cost estimation for additional structures***

a. Estimated cost of reinforcement work.
   There is a difference between the calculations using conventional methods and BIM-based methods. The difference was 52.59 kg, representing 1.665%. The BIM-based method calculation indicates that the work cost was IDR 64,983,375.45, which accounts for 40.693% of the total project cost.
b. Estimated cost of concrete work.
   The volume difference of concrete between the conventional and BIM-based methods is 0.59 m³, representing 3.576%. The BIM-based method calculation shows that the work cost was IDR 16,353,676.47, which accounts for 10.241% of the total project cost.
c. Estimated cost of formwork.
   The volume difference for formwork between the conventional and BIM-based methods is 4,901 m², representing a percentage difference of 2,624%. The BIM-based method calculation indicates that the cost for formwork was IDR 61,861,049.77, accounting for 38.738% of the total project cost.

***Estimated total cost***

Based on all cost estimations, the total cost for renovating a residential house into a shophouse is IDR 159,692,607.93. Details are provided in Table 11.

Table 10. Cost estimation

| Work List | Cost Estimation |
|---|---|
| Wall demolition | IDR 10.116.811,17 |
| Roof demolition | IDR 6.366.994,45 |
| Column concrete cover demolition | IDR 10.700,62 |
| Reinforcement | IDR 64.983.375,45 |
| Concrete | IDR 16.353.676,47 |
| Formwork | IDR 61.861.049,77 |
| **Total** | **IDR 159.692.607,93** |

***Time scheduling***

a. First week progress
In the first week, the roof demolishing was completed, as in Figure 11.

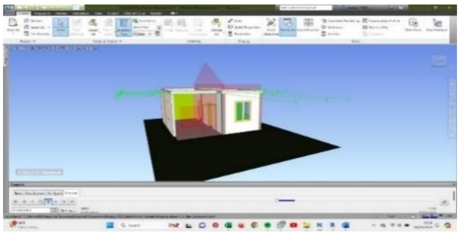

Figure 11. First week progress

b. Second week progress
By the second week, the demolition work was completed, and reinforcement work for the first-floor columns began, as shown in Figure 12.

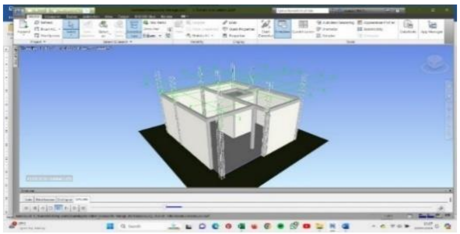

Figure 12. Second week progress

c. Third week progress
In the third week, the reinforcements for the first-floor columns were installed, and formwork and casting began, as shown in Figure 13.

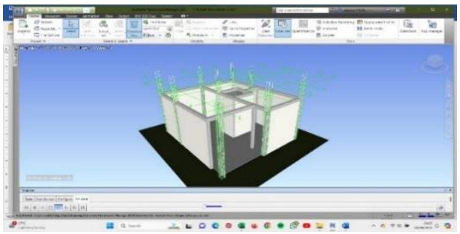

Figure 13. Third week progress

d. Fourth week progress
In the fourth week, the reinforcement for the first-floor columns was completed, and work began on reinforcing the second-floor beams and slab, as shown in Figure 14.

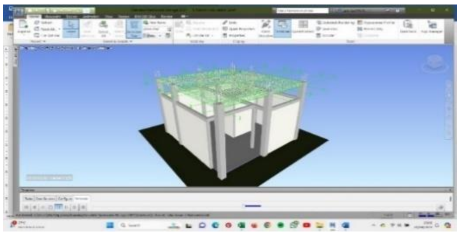

Figure 14. Fourth week progress

e. Fifth week progress
In the fifth week, reinforcement and formwork for the second-floor beams and slabs were completed, allowing casting to proceed, as shown in Figure 15.

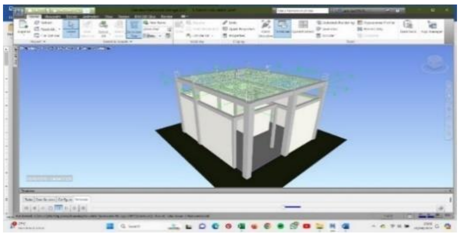

Figure 15. Fifth week progress

f. Sixth week progress
In the sixth week, the second-floor slab and beams were completed, as shown in Figure 16.

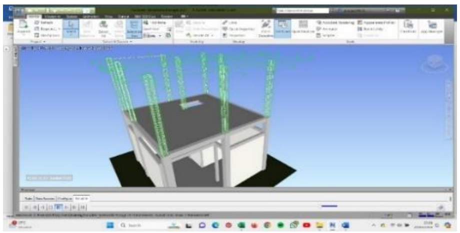

Figure 16. Sixth week progress

g. Seventh week progress
In the seventh week, the second-floor columns were completed, and reinforcement for the rooftop beams and slab began, as shown in Figure 17.

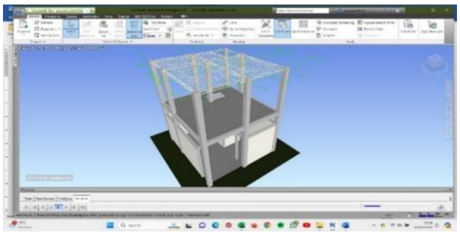

Figure 17. Seventh week progress

h. Eighth week progress
In the eighth week, formwork was installed on the rooftop beams and floor slabs, allowing concrete casting to begin, as shown in Figure 18.

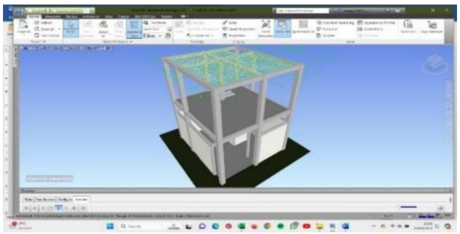

Figure 18. Eighth week progress

i. Day 58 progress
On the 58th day, all structural work for renovating the residential house into a two-story shophouse was completed, as shown in Figure 19.

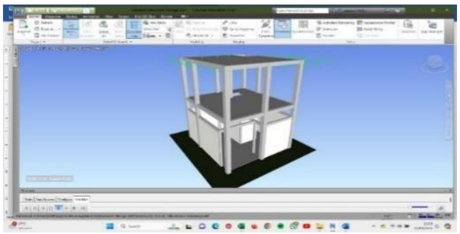

Figure 19. Ninth week progress

## Conclusions

The evaluation of the existing structure using ASCE 41-17 revealed deficiencies in global strength and component ductility, indicating the need for structural retrofitting. FEMA 547 offers a solution to these issues through the application of concrete jackets for retrofitting.

Before retrofitting, the existing structure, with the additional floors, exhibited capacity ratios greater than 1 and deflections exceeding permitted limits. However, after retrofitting with concrete jackets, the capacity ratio significantly decreased, with the highest value being 0.611. Additionally, the deflections were within permissible limits, with the maximum deflection recorded at 34.832 mm.

The volume calculated by Revit is highly dependent on the accuracy of the 3D design. The estimated cost calculation using Autodesk Revit indicates that demolition work amounts to IDR 16,494,506.24, representing 10.26% of the total cost. Structural work is estimated at IDR 143,198,101.69, accounting for 89.67% of the total cost. Therefore, the total estimated cost for the structural renovation of a residential house into a shophouse is IDR 159,692,607.93.

The duration of the work, as determined using Autodesk Navisworks, is 58 days. This includes 11 days allocated for demolition work and 47 days for the construction of the structural work.

Based on this research, it is suggested that the data used to evaluate existing structures with ASCE 41-17 should be more comprehensive to enhance the accuracy of the assessments and retrofitting recommendations. Additionally, when using BIM-based software, employing high-specification devices is crucial to avoid delays and ensure smooth operation during 3D design work.

## References

American Concrete Institute. (2019). Building Code Requirements for Structural Concrete. Farmington Hills: American Concrete Institute.

American Society of Civil Engineers. (2017). ASCE/SEI 41-17 Seismic Evaluation and Retrofit of Existing Buildings. Reston: American Society of Civil Engineers.

Badan Standardisasi Nasional. (2019). SNI 1726:2019 Tata Cara Perencanaan Ketahanan Gempa untuk Struktur Bangunan Gedung dan Nongedung. Jakarta: Badan Standardisasi Nasional.

Badan Standardisasi Nasional. (2019). SNI 2847-2019 Persyaratan Beton Struktural untuk Bangunan Gedung dan Penjelasan. Jakarta: Badan Standardisasi Nasional.

Badan Standardisasi Nasional. (2020). SNI 1727-2020 Beban Desain Minimum dan Kriteria Terkait untuk Bangunan Gedung dan Struktur Lain. Jakarta: Badan Standardisasi Nasional.

FEMA. (2006). Technique for Seismic Rehabilitation of Existing Buildings FEMA 547. Washington, D.C: Federal Emergency Management Agenc.