COMPARATIVE ANALYSIS OF PRECAST CONCRETE FLOOR SLAB WITH CAST IN SITU AND STRUCTURE STABILITY OF GKB UPN "VETERAN" EAST JAVA

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Abstract

_Gedung Kuliah Bersama_ was modified into ten floors, constructed using precast pretension slabs, namely hollow core slabs on the second to five floors and cast in situ slabs, namely steel deck on floors six to ten. _Gedung Kuliah Bersama_ is located at the UPN “Veteran” East Java in the city of Surabaya. Geologically and tectonically, Surabaya City is in an active fault zone, so that the tectonic activity that occurs can cause damage to building structures and construction materials. Pushover analysis is an analysis that can be used to determine the pattern of structural collapse when an earthquake occurs. Based on the results of the analysis, precast pretension slabs (hollow core slabs) with segments of 1500 mm × 7000 mm × 150 mm with PC Wire reinforcement φ-7-121 mm and cast in situ (steel deck) slabs with a thickness of 120 mm with wiremesh reinforcement M8-150 mm. The difference in the bending strength of the two types of slabs is 38.73%. The results of the pushover analysis show that the structural performance at the Damage Control (DO) level shows that the building is able to withstand the earthquake that occurs and the risk of casualties is very small.

Keywords: precast pretension, cast in situ, pushover, structure stability

INTRODUCTION

_Gedung Kuliah Bersama_ UPN “Veteran” East Java is a building that has a length of 42 m, a width of 24 m and a height of 24 m. The _Gedung Kuliah Bersama_ UPN “Veteran” East Java functions as an educational facility and has six floors in which precast pretension slabs are constructed, namely hollow core slabs on the second to third floors and cast in situ slabs, namely steel deck on floors four to six. Then in this study, the modification was carried out into ten floors using two types of floor slabs, namely precast pretension slabs on floors two to five and cast in situ slabs on floors six to ten. The height between floors is 4 m.

Precast concrete is a structural element that is molded at a certain place (factory) which is then brought to the project site and installed on the structure. The process of producing precast concrete is usually carried out through repeated mass production with the shape and size according to the order (Najoan et al. 2016: 320). Cast in situ concrete is the process of casting concrete by moving the liquid concrete mixture from the mixer to the location of the concrete to be cast, namely formwork or a reference to the structure to be worked on (Najoan et al. 2016: 321).

_Gedung Kuliah Bersama_, located at UPN “Veteran” East Java, is included in the Surabaya City area, which is geologically and tectonically located in an active fault zone, namely the Baribis-Kendeng zone based on the Source and Hazard Map of the 2017 Indonesian Earthquake. If activity occurs on this active fault can cause earthquakes on land that have the potential to cause damage to structures and construction materials (Daryono et al., 2009: 596). Therefore, it is necessary to analyze the stability of the structure to determine the pattern of structural collapse when an earthquake occurs so that it can reduce material losses and casualties.

Pushover analysis is an analysis that can be used to determine the pattern of structural collapse when an earthquake occurs. Pushover analysis or static thrust load analysis is an analysis by increasing the static load in the lateral direction whose value is increased gradually proportionally to the structure until it reaches the displacement target or reaches a mechanism on the verge of collapse (ATC-40, 1996: xiii). Through pushover analysis, it can be seen which parts of the building will experience collapse first when an earthquake occurs.

Based on these circumstances, encouraging the authors to analyze the comparative design of pretension and cast in situ slabs in terms of slab thickness, the reinforcement used and the performance of each slab. An analysis of the stability of the frame structure of the _Gedung Kuliah Bersama_ UPN "Veteran" East Java was also carried out with software.

The aim of this research is to know the analysis of precast pretension floor slabs, analysis of cast in situ concrete floor slabs, comparative results of slab analysis, frame structure performance, and analysis of the ductility of the frame structure at the _Gedung Kuliah Bersama_ UPN "Veteran" East Java.
RESEARCH METHOD

Based on the literature review that has been compiled, a flowchart can be made to simplify the research process as shown in Figure 1.

A. Flowchart Explanation

1. Secondary Data Collection
   Collecting data needed at the Gedung Kuliah Bersama UPN "Veteran" East Java such as shop drawings, technical specifications, and soil data.

2. Preliminary Design
   Preliminary design includes calculating the dimensions of each structural component. The following is the calculation of the preliminary design of beam structural components:
   a. The minimum height of non-prestressed beams for simple attachment is \( \frac{L}{16} \) (table 9.3.1.1, SNI 2847:2019).
      \[ h_{\text{min}} = \frac{L}{16} \quad (1) \]
   b. The minimum height of non-prestressed beams for cantilever attachment is \( \frac{L}{8} \) (table 9.3.1.1, SNI 2847:2019).
      \[ h_{\text{min}} = \frac{L}{8} \quad (2) \]
   The following is the calculation of the preliminary design of column structure components:
   \[ l_{\text{column}} = \frac{h_{\text{column}}}{h_{\text{beam}}} \quad (3) \]
   Here is the calculation to determine the slab thickness:
   a. The minimum thickness of the one way slab
      The minimum thickness of the one way slab is shown in Table 8.3.1.1 SNI 2847:2019.
      a. Minimum thickness of two way slab (Clause 8.3.1.2 SNI 2847:2019)
         1) For \( \alpha_m < 0.2 \)
            \[ h \geq 120 \text{ mm} \quad (4) \]
         2) For \( 0.2 < \alpha_m < 2 \)
            \[ h = \frac{\ln (0.8 + \frac{f_y}{1500})}{36 + 5\beta (\alpha_m - 0.2)} \geq 120 \text{ mm} \quad (5) \]
         3) For \( \alpha_m > 2 \)
            \[ h = \frac{\ln (0.8 + \frac{f_y}{1500})}{36 + 9\beta} \geq 90 \text{ mm} \quad (6) \]
   Information:
   \( \alpha_m \) = the average value of the bending stiffness ratio beam cross section against strength bending slab.
   \( \beta \) = net span ratio in directions extends against the direction retract from the slab.

3. Calculation of Dead Load and Live Load
   The dead load calculation in this study is based on PPIUG 1983 and live load based on SNI 1727: 2013.

4. Earthquake Load Calculation
   Earthquake loads were analyzed using response spectrum analysis in accordance with Clause 6 of SNI 1726: 2019. Before calculating the design response spectrum, it is necessary to calculate the following periods:
   1. \( T_0 = 0.2 \times \frac{S_{DI}}{S_{DS}} \quad (7) \]
   2. \( T_S = \frac{S_{DI}}{S_{DS}} \quad (8) \]
   3. The \( T_S \) value is obtained based on the long period transition map on SNI 1726:2019.
   The provisions in determining the design acceleration response spectrum (\( S_a \)) based on Clause 6.4 SNI 1726:2019 are as follows:
   1. For periods smaller than \( T_0 \)
      \[ S_a = S_{DS} \times \left( 0.4 + 0.6 \times \frac{T}{T_0} \right) \quad (9) \]
2. For periods greater than or equal to \( T_0 \) and less than or equal to \( T_s \)
   \[ S_a = S_{PS} \quad (10) \]
3. For periods greater than \( T_s \), but less than or equal to \( T_L \)
   \[ S_a = \frac{S_{PS}}{T} \quad (11) \]
4. For periods greater than \( T_L \)
   \[ S_a = \frac{S_{PS} \times t_L}{T^2} \quad (12) \]

5. **Precast Slab Analysis**
   The calculation of the hollow core slab that is carried out includes material planning, calculation of prestress force, loss of prestress, checking for stress, cracks, shear, and deflection.

5.1 **Cross-Sectional Area and Inertia**
   The cross section force of the hollow core slab is shown in Figure 2.

![Cross Section Force and Representative Volume of Hollow Core Slab](source: P.C.J. Hoogenboom, 2005.)

Here are the steps for calculating the cross-sectional area and inertia of the hollow core slab:
1. Area of the flange (a)
   \[ a = \frac{b_1}{2} \times (h - t_1 - t_2) \quad (13) \]
2. Area per unit length of section in the direction of the x axis (\( A_x \))
   \[ A_x = t_1 + t_2 + a \quad (14) \]
3. Area of concrete per unit length of section on the x axis (\( A_{C_x} \))
   \[ A_{C_x} = A_x \times bw \quad (15) \]
4. The x axis elastic modulus (\( S_x \))
   \[ S_x = \frac{1}{2} \times t_1^3 + a \times \frac{1}{2} \times (h + t_1 - t_2) + t_2 \times (h - \frac{1}{2} \times t_2) \quad (16) \]
5. The x axis plastic modulus (\( Z_x \))
   \[ Z_x = \frac{S_x}{E_x} \quad (17) \]
6. The moment of inertia per unit length of the section in the direction of the x axis (\( I_x \))
   \[ I_x = \frac{1}{12} \times t_1^3 + t_1 \times (Z_x - \frac{1}{2} \times t_1)^2 + \frac{1}{12} \times t_2^3 + t_2 \times (h - Z_x - \frac{1}{2} \times t_2)^2 + \frac{1}{12} \times a \times (h - t_1 - t_2)^2 + a \times (Z_x - \frac{1}{2} \times (h + t_1 - t_2))^2 \quad (18) \]
   \[ I_{C_x} = I_x \times bw \quad (19) \]
7. Area per unit length of section in the direction of the y axis (\( A_y \))
   \[ A_y = t_1 + t_2 \quad (20) \]
8. Area of concrete per unit length of section on the y axis (\( A_{C_y} \))
   \[ A_{C_y} = A_y \times Lsl \quad (21) \]
9. The y axis elastic modulus (\( S_y \))
   \[ S_y = \frac{1}{2} \times t_2^3 + t_1 \times (h - \frac{1}{2} \times t_2) \quad (22) \]
10. The y axis plastic modulus (\( Z_y \))
    \[ Z_y = \frac{S_y}{E_y} \quad (23) \]
11. The moment of inertia per unit length of the section in the direction of the y axis (\( I_y \))
    \[ I_y = \frac{1}{12} \times t_1^3 + t_1 \times (Z_y - \frac{1}{2} \times t_1)^2 + \frac{1}{12} \times t_2^3 + t_2 \times (h - Z_y - \frac{1}{2} \times t_2)^2 \quad (24) \]
    \[ I_{C_y} = I_y \times Lsl \quad (25) \]

5.2 **Prestressed Concrete Planning**
There are several limitations in concrete planning, including the following:

a. Limitation of the stress of concrete in the prestressed distribution conditions (transfer stage).
   Limitation of compressive stress (Clause 24.5.3.1, SNI 2847:2019)
   \[ f_{ci} \leq 0.6 \times f_{ci}' \quad (26) \]
   Limitation of tensile stress (Clause 24.5.3.2, SNI 2847:2019)
   \[ f_t = 0.25 \times \sqrt{f_{ci}} \quad (27) \]

b. Limitation of concrete stress in service load conditions (Table 24.5.4.1, SNI 2847:2019)
   (prestress + dead load)
   \[ f_c = 0.45 \times f_{ci}' \quad (28) \]
   (prestress + total load)
   \[ f_t = 0.6 \times f_t' \quad (29) \]
   Based on SNI 2847:2019, Clause 19.2.2.1 the formula for modulus of elasticity (\( E_c \)) for normal concrete is as follows:
   \[ E_c = 4700 \times \sqrt{f_{ci}} \quad (30) \]
   Information:
   \[ f_{ci}' = \text{compressive strength of concrete at 7 days of age (MPa).} \]
   \[ f_t' = \text{compressive quality (MPa).} \]

5.3 **Prestressed Steel Planning**
Based on table 20.3.2.5.1 SNI 2847:2019 there is a limit to the tensile stress of prestressed steel, namely as follows:

a. At the stressing stage, the jacking tip is
   \[ 0.94 \times f_{py} \leq 0.80 \times f_{pu} \]
   \[ 0.94 \times f_{py} \quad (31) \]
   \[ 0.80 \times f_{pu} \quad (32) \]

b. At the post transfer stage in the post cataric anchor and couplers is 0,7
   \[ 0.70 \times f_{pu} \quad (33) \]

5.4 **Stress in Prestressed Concrete**
The following is the calculation of stress in prestressed concrete according to Edward G. Nawy (2001):

a. Transfer stage (when applying the prestress force)
   Top fiber tension:
   \[ f_1 = \frac{P_t}{A_e} \times \left( 1 - \frac{e \times C}{r^2} \right) - \frac{M_0}{s_t} \quad (34) \]
   \[ f_1 \leq f_{ci} \quad (35) \]
5.5 Prestressed Force Losses
In the prestressed pretension concrete structure, the loss of the prestressed force is calculated due to the elastic shortening of the concrete, concrete creep, and concrete shrinkage.

a. Shortening of Elastic Concrete (ES)
\[ \Delta f_{pES} = n \times \bar{f}_{cs} \]  
\[ n = \frac{E_s}{E_{ci}} \]  
\[ \bar{f}_{cs} = -\frac{P_i}{A_{cs}} \left( 1 + \frac{e^2}{r^2} \right) + \frac{M_{D} E_{ci}}{I_{c}} \]  

b. Concrete Creep (CR)
\[ \Delta f_{pCR} = K_{CR} \times \frac{E_s}{E_{ci}} \times (\bar{f}_{cs} - \bar{f}_{csd}) \]  
\[ \bar{f}_{csd} = \frac{M_{D} E_{ci}}{I_{c}} \]  
Information:
\[ K_{CR} = 2 \] (pretension structural components)  
\[ 1.6 \] (posttension structural components)

c. Concrete Shrinkage (\( \Delta f_{PSH} \))
\[ \Delta f_{PSH} = K_{sh} \times 8.2 \times 10^{-6} \times \left( 1 - 0.06 \times \frac{V}{S} \right) \times (100 - RH) \times E_s \]  
Information:
\[ RH = 75 \% \] (Source: Meteorology Station Class I Juanda Surabaya)
\[ K_{sh} = 1 \] (pretension)

5.6 Deflection Check
a. Deflection due to prestress force (\( \Delta p_i \))
\[ \Delta p_i = -\frac{P_i x e x L s_{p} l^2}{8x E c i x l c} \]  

b. Deflection due to dead load (\( \Delta b_s \))
\[ \Delta b_s = \frac{5x Q_{s} a b x L s_{p} l^4}{384x E c i x l c} \]
6. Reinforcement calculation
   a. Reinforcement area needed (As needed)
      As needed = \( \rho \times b \times d \) (mm\(^2\))  
      (62)
   b. Reinforcement area used (As used)
      As used = \( \frac{1}{4} \pi d^2 \) (mm\(^2\))  
      (63)
   c. Number of reinforcement (n)
      \( n = \frac{\text{As needed}}{\text{As used}} \)  
      (64)
   d. Reinforcement distance (S)
      \( S = \frac{b}{n} \)  
      (65)
   e. Maximum reinforcement distance (S max)
      S max = 2 \times h slab  
      (66)

7. Steel Deck Slab Analysis

Steel deck slab analysis includes steel deck and wiremesh planning. The following is the calculation of steel deck according to the Steel Deck Institute 2011:

1. Distance from top of concrete to center of steel deck (d)
   \( d = h - \frac{1}{2} \times \text{wave height} \)  
   (67)
2. Concrete depth over steel deck (hc)
   \( hc = h - \text{wave height} \)  
   (68)
3. Distance from top of slab to neutral line of cracked section (Ycc)
   \( Ycc = d \left( \sqrt{2\rho n + (\rho m)^2} - \rho m \right) < hc \)  
   (69)
   \( Ycs = d - Ycc \)  
   (70)
4. The ratio of modulus of elasticity of steel and concrete (n)
   \( n = \frac{E_s}{E_c} \)  
   (71)
   \( Ec = 4700 \sqrt{f_c'} \) (Mpa)  
   (72)
5. Reinforcement ratio (\( \rho \))
   \( \rho = \frac{As}{b \times d} \)  
   (73)
6. Moment of inertia of the cracked section (Ic)
   \( Ic = \frac{b}{3} \times Ycc^3 + As \times Ycs^2 + Isf \)  
   (mm\(^2\))  
   (74)
   \( Isf = \text{moment of inertia of full steel deck per unit} \)  
   (mm\(^2\))

7. Calculate bending strength
   \( My = \frac{Fy \times x}{b - Ycc} \)  
   (75)
   \( Mru = \Phi \times My \)  
   (76)

Information:
\( h = \text{slab thickness (mm)} \)
\( \Phi = 0.85 \)
\( Fy = \text{steel deck yield (Mpa)} \)

Check the requirements:
Mru > Mu+
The following is the calculation of wiremesh according to Dewi dan Kusmila, 2018:

1. Coventional reinforcement
   \( As = \frac{1}{4} \times \pi \times d^2 \times \left( \frac{1000}{8} \right) \)  
   (77)
2. Wiremesh reinforcement

As needed = \( As \times \left( \frac{f_y}{f_yw} \right) \)  
(78)
As w = \( \frac{1}{4} \times \pi \times d^2 \times \left( \frac{1000}{8} \right) \)  
(79)
If As w > As needed (OK)

3. Number of wiremesh (n wiremesh)
   \( n \text{ wiremesh} = \frac{\text{area of floor slab}}{\text{area of 1 wiremesh}} \)  
   (80)

7. Pushover Analysis

From the displacement value in the pushover analysis results, it will be known the seismic performance criteria of the structure based on ATC-40. The limitations of the roof drift ratio according to ATC-40 are shown in Table 1.

Table 1 Limitation of Roof Drift Ratio

<table>
<thead>
<tr>
<th>Displacement Limits Between Levels</th>
<th>Structure Performance</th>
</tr>
</thead>
<tbody>
<tr>
<td>IO</td>
<td>DO</td>
</tr>
<tr>
<td>Maximum total displacement</td>
<td>0,01</td>
</tr>
<tr>
<td>Maximum inelastic displacement</td>
<td>0,005</td>
</tr>
</tbody>
</table>

(Source: ATC-40, 1996)

RESULT AND DISCUSSION

A. Preliminary Design

The planning data used are as follows:

a. Building function = education facilities
b. Building length = 42 m
c. Building width = 24 m
d. Building height = 40 m
e. Number of floors = 10 lantai
f. Height of each floor = 4 m
g. Quality of cast in situ concrete (\( f_c' \)) = 25 MPa
h. Quality of precast concrete (\( f_c' \)) = 40 MPa
i. Quality of plain bar (\( f_y \)) = 240 MPa
j. Quality of deformed bar (\( f_y \)) = 410 MPa

Table 2 Recapitulation of Preliminary Design

<table>
<thead>
<tr>
<th>Number</th>
<th>Component Structure</th>
<th>Dimension (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>b \times h \times L</td>
<td></td>
</tr>
<tr>
<td>1.</td>
<td>B1</td>
<td>500</td>
</tr>
<tr>
<td>2.</td>
<td>B2</td>
<td>300</td>
</tr>
<tr>
<td>3.</td>
<td>BA</td>
<td>200</td>
</tr>
<tr>
<td>4.</td>
<td>BK</td>
<td>150</td>
</tr>
<tr>
<td>5.</td>
<td>K1</td>
<td>1000</td>
</tr>
<tr>
<td>6.</td>
<td>K2</td>
<td>300</td>
</tr>
</tbody>
</table>

Thickness (mm)

7. Floor Slab 120
8. Roof Slab 100
9. Luifel Slab 100

(Source: Calculation Results)

B. The Load of Structure

The load calculation does not included wind loads, rain loads and snow loads. The load combinations used are as follows:
1. 1.4D
2. 1.2D + 1.6L
3. 1.2D + 1L
4. 1.2D + 1.6L + 0.5Lr
5. 1.4D + 1.6Lr + 1L
6. 1.2D + 1.0EQx + 0.3EQy + 0.5L
7. 1.2D + 1.0EQy - 0.3EQx + 0.5L
8. 1.2D - 1.0EQx + 0.3EQy + 0.5L
9. 1.2D + 1.0EQy + 0.3EQx + 0.5L
10. 1.2D - 1.0EQy - 0.3EQx + 0.5L
11. 1.2D + 1,0EQy - 0.3EQx + 0.5L

1. **Dead Load Analysis**

Dead load recapitulation is shown in Table 3.

<table>
<thead>
<tr>
<th>Number</th>
<th>Dead Load</th>
<th>Weight (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Slab weight</td>
<td>0.000024</td>
</tr>
<tr>
<td>2.</td>
<td>Ceiling</td>
<td>0.00018 MPa</td>
</tr>
<tr>
<td>3.</td>
<td>Space per cm</td>
<td>0.00021 MPa</td>
</tr>
<tr>
<td>4.</td>
<td>Tile per cm</td>
<td>0.00024 MPa</td>
</tr>
<tr>
<td>5.</td>
<td>MEP</td>
<td>0.00020 MPa</td>
</tr>
</tbody>
</table>

(Table Source: PPIUG 1983)

2. **Live Load Analysis**

Live loads that occur on floors 1-10 are shown in Table 4.

<table>
<thead>
<tr>
<th>Number</th>
<th>Type of Load</th>
<th>Weight (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>First Floor Corridor</td>
<td>0.00479</td>
</tr>
<tr>
<td>2.</td>
<td>Corridor above the First Floor</td>
<td>0.00383</td>
</tr>
<tr>
<td>3.</td>
<td>Classroom</td>
<td>0.00192</td>
</tr>
<tr>
<td>4.</td>
<td>Office Room</td>
<td>0.00240</td>
</tr>
<tr>
<td>5.</td>
<td>Laboratory Room</td>
<td>0.00287</td>
</tr>
<tr>
<td>6.</td>
<td>Meeting Room</td>
<td>0.00479</td>
</tr>
<tr>
<td>7.</td>
<td>Flat Roof</td>
<td>0.00096</td>
</tr>
</tbody>
</table>

(Table Source: Table 4-1 SNI 1727:2013)

3. **Earthquake Load Analysis Response Spectrum Method**

Earthquake loads that work on the building structure are planned according to the location of the Gedung Kuliah Bersama UPN "Veteran" East Java, located in the city of Surabaya, East Java. The calculations used the provisions of SNI 1726: 2019. The spectrum response curve is shown in Figure 4.

Figure 4 Response Spectrum of Surabaya City Design for SE Site Class (Source: Calculation Results)

C. **Check The Output Analysis Results**

1. Check the fundamental period of the structure
   \[ Tc = 1,396 \text{ seconds} < T = 1,805 \text{ seconds} \text{(OK)} \]
2. Check variant combinations
   For a difference of \( T < 15\% \), a combination of CQC variants was used
3. Check mass participation
   \[ x = 99\%, \ y = 99\% \text{ (OK)} \]
4. The value of shear force
   \[ x = 10099799.73 \text{ N}, \ y = 8764379.18 \text{ N} \]
5. Check floor displacements
   Allowance displacement 30,769 mm (OK)

D. **Conventional Slabs**

Based on the calculations that have been done, the results are shown in Table 5.

<table>
<thead>
<tr>
<th>Number</th>
<th>Slab Type</th>
<th>Reinforcement Types</th>
<th>Calculation Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Floor slab</td>
<td>Field ∅10-125 mm</td>
<td></td>
</tr>
<tr>
<td>2.</td>
<td>Roof slab</td>
<td>X axis field ∅10-200 mm</td>
<td></td>
</tr>
<tr>
<td>3.</td>
<td>Luifel slab</td>
<td>Field ∅120-200 mm</td>
<td></td>
</tr>
</tbody>
</table>

(Table Source: Calculation Results)

E. **Hollow Core Slab dan Steel Deck**

Based on the calculations that have been done, the results are shown in Table 6.

<table>
<thead>
<tr>
<th>Number</th>
<th>Comparative Analysis of Slabs</th>
<th>Hollow Core Slab (Precast)</th>
<th>Steel Deck (Cast In Situ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Slab thickness 150 mm</td>
<td>120 mm</td>
<td></td>
</tr>
<tr>
<td>2.</td>
<td>Concrete quality 40 MPa</td>
<td>25 MPa</td>
<td></td>
</tr>
<tr>
<td>3.</td>
<td>Reinforcement quality 1377 MPa</td>
<td>500 MPa</td>
<td></td>
</tr>
<tr>
<td>4.</td>
<td>reinforcement PC Wire ∅7-121 mm</td>
<td>Wiremesh M8-150 mm</td>
<td></td>
</tr>
<tr>
<td>5.</td>
<td>The sectional limit bending strength</td>
<td>65441950 N.mm</td>
<td>40999500 N.mm</td>
</tr>
</tbody>
</table>

(Table Source: Calculation Results)
Based on the calculations that have been done, the results are shown in Table 7 and Table 8.

Table 7 Recapitulation of Beam Reinforcement

<table>
<thead>
<tr>
<th>Number.</th>
<th>Calculation Results</th>
<th>B1 (50/70)</th>
<th>B2 (35/50)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Tension reinforcement</td>
<td>10D29</td>
<td>6D25</td>
</tr>
<tr>
<td>2.</td>
<td>Stress reinforcement</td>
<td>5D29</td>
<td>4D25</td>
</tr>
<tr>
<td>3.</td>
<td>Shear reinforcement inside joint</td>
<td>2D13 - 90 mm</td>
<td>2φ10 - 80 mm</td>
</tr>
<tr>
<td>4.</td>
<td>Shear reinforcement outside joint</td>
<td>2D13 - 200 mm</td>
<td>2φ10 - 150 mm</td>
</tr>
<tr>
<td>5.</td>
<td>Torsion reinforcement</td>
<td>2D29</td>
<td>-</td>
</tr>
<tr>
<td>6.</td>
<td>Length of distribution tension reinforcement</td>
<td>900 mm</td>
<td>900 mm</td>
</tr>
<tr>
<td>7.</td>
<td>Length of distribution stress reinforcement</td>
<td>600 mm</td>
<td>600 mm</td>
</tr>
</tbody>
</table>

(Source: Calculation Results)

Table 8 Recapitulation of Beam Reinforcement (Continue)

<table>
<thead>
<tr>
<th>Number.</th>
<th>Calculation Results</th>
<th>BA (20/30)</th>
<th>BK (15/20)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Tension reinforcement</td>
<td>4D22</td>
<td>2φ10</td>
</tr>
<tr>
<td>2.</td>
<td>Stress reinforcement</td>
<td>2D22</td>
<td>2φ10</td>
</tr>
<tr>
<td>3.</td>
<td>Shear reinforcement inside joint</td>
<td>2φ10 - 50 mm</td>
<td>2φ8 - 50 mm</td>
</tr>
<tr>
<td>4.</td>
<td>Shear reinforcement outside joint</td>
<td>2φ10 - 75 mm</td>
<td>2φ8 - 75 mm</td>
</tr>
<tr>
<td>5.</td>
<td>Length of distribution tension reinforcement</td>
<td>700 mm</td>
<td>300 mm</td>
</tr>
<tr>
<td>6.</td>
<td>Length of distribution stress reinforcement</td>
<td>450 mm</td>
<td>200 mm</td>
</tr>
</tbody>
</table>

(Source: Calculation Results)

Table 9. Recapitulation of Column Reinforcement

<table>
<thead>
<tr>
<th>Number.</th>
<th>Calculation Results</th>
<th>K1 (100/100)</th>
<th>K2 (30/30)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Reinforcement ratio</td>
<td>1%</td>
<td>2%</td>
</tr>
<tr>
<td>2.</td>
<td>Bending reinforcement</td>
<td>16D32</td>
<td>16D22</td>
</tr>
<tr>
<td>3.</td>
<td>Shear reinforcement inside joint</td>
<td>4D22 - 100 mm</td>
<td>2D13 - 100 mm</td>
</tr>
<tr>
<td>4.</td>
<td>Shear reinforcement outside joint</td>
<td>4D22 - 150 mm</td>
<td>2D13 - 150 mm</td>
</tr>
<tr>
<td>5.</td>
<td>Length of distribution reinforcement</td>
<td>1250 mm</td>
<td>900 mm</td>
</tr>
</tbody>
</table>

(Source: Calculation Results)

G. Joint of Column and Beam

Reinforcement at the joint is as follows:
K1 = D22 – 100 mm
K2 = D13 – 100 mm

H. Pushover Analysis

The results obtained from the pushover nonlinear static analysis are expected to be able to show the performance of the building structure being tested by being given continuous earthquake loads. Pushover analysis is carried out by entering the calculation of reinforcement in the auxiliary program to check the performance of the calculated structure.

1. X Axis Output

The x axis pushover curve is shown in Figure 7.

(Source: Software Analysis)

Based on Figure 7, the following values are obtained:

a. Base Shear (V) = 35320237 N
b. Displacement (D) = 277,371 mm
c. Effective Period (Teff) = 1,525 detik (step ke-4)
d. Effective Damping (Beff) = 0,173 ≈ 17,3%

Here is the determination of structure performance level:
1. Maximum total displacement (ATC-40)

\[
\frac{\Delta H_{\text{tot}}}{\Delta H_{\text{tot}}} = \frac{298,68}{40000} = 0,007 \leq 0,01 \text{ (IO)}
\]
2. Maximum inelastic displacement
\[
\delta = \frac{\delta_{\text{second yield}} - \delta_{\text{first yield}}}{H}
\]
\[
= \frac{107,539 - 42,176}{40,000}
= 0.0016 \leq 0.005 (DO)
\]
The ductility factor of x axis (\(\mu_x\)) can be calculated by the following equation:
\[
\mu_x = \frac{277,371}{42,176} = 6.577
\]
2. X Axis Output
The y axis pushover curve is shown in Figure 8.

Figure 8 The Y Axis Pushover Curve
(Source: Software Analysis)

Based on Figure 8, the following values are obtained:

a. Base Shear (V) = 31498337 N
b. Displacement (D) = 295,968 mm
c. Effective Period (Teff) = 1.69 detik (step ke-4)
d. Effective Damping (Befl)= 0.172 \approx 17.2\%

Here is the determination of structure performance level:

1. Maximum total displacement (ATC-40)
\[
\frac{D}{H} = \frac{584,157}{40,000} = 0.0146
\rightarrow 0.01 \sim 0.02 (DO)
\]
2. Maximum inelastic displacement
\[
\frac{\delta}{H} = \frac{111,799 - 13,715}{40,000}
= 0.001 \leq 0.005 (DO)
\]
The ductility factor of y axis (\(\mu_y\)) can be calculated by the following equation:
\[
\mu_y = \frac{295,968}{72,715} = 4.07
\]

CONCLUSION

Based on the results of the analysis at the Gedung Kuliah Bersama UPN “Veteran” East Java, the following conclusions are:

1. The comparison between the precast pretension slab (hollow core slab) and the cast in situ slab (steel deck) shows that there is a difference in the value of the bending strength of the cross-sectional limit is 38.73%.
2. The capacity spectrum method uses pushover analysis with the category of building performance levels that occur through the maximum deviation value. The x axis pushover output produces displacement target is 277,371 mm, shear force is 35320237 N, effective period is 1.525 seconds, and damping is 17.3%.
Meanwhile, the y axis pushover output produces target displacement is 295.968 mm, shear force is 31498337 N, effective period is 1.69 seconds, damping is 17.2%.
So, the level of performance of the structure of the Gedung Kuliah Bersama UPN “Veteran” East Java is Damage Control (DO).
3. Based on the results of the ductility analysis, the ductility factor value is 6.577 in the x axis and 4.07 in the y axis. So, the Gedung Kuliah Bersama UPN “Veteran” East Java is a partial ductile structure.

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REFERENCES


