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Assessment and retrofitting of nursing faculty building of Andalas University, Padang, Indonesia

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Abstract. The Faculty of Nursing building is one of the administrative buildings at Andalas University. This building was designed by a planning consultant and began construction in 2013. However, during the construction period, it was found that the concrete quality was very low, that is fc'=14.32 MPa, so that the construction could not be continued because it was not in accordance with the planning quality, fc' =22.39 MPa. Therefore, it is necessary to evaluate the feasibility of the building structure before the construction is continued. In the structural analysis, the loads apllied are dead loads, live loads, and earthquake loads. The analysis was carried out using the application *(software)* ETABS. Based on the analysis results, it was found that the capacity of columns and beams of the building are not strong enough to withstand the loads acting on the structure. The inter story drift also does not meet the permit limit requirements according to the New Indonesian Earthquake Code, SNI 1726:2019. Therefore, it is necessary to retrofit (strengthen) the structure of the Nursing Faculty Building by re-designing the Detail Engineering Design (DED) on the beams and columns of the plan and jacketing the existing columns. Re-analysis results show that the retrofitted building structure has a strong enough capacity to carry loads acting on the structure and the inter story drift has met the permit limit requirements according to the SNI 1726:2019.

1 Introduction

The Faculty of Nursing building is one of the administrative buildings at Andalas University. This building was designed by a planning consultant and began construction in 2013. However, during the construction period, it was found that the concrete quality was very low, that is fc'=14.32 Mpa, so that the construction could not be continued because it was not in accordance with the planning quality, fc' =22.39 MPa [1]. The demolition of the building certainly requires a large amount of money. Therefore, it is necessary to have an action to retrofit (strengthen) the structure so that the building can carry the working load.

Therefore, it is necessary to assess of the structure and design retrofitting the structure of the Nursing Faculty Building, Andalas University.

2 Evaluation of Existing Structure

2.1 Condition of Existing Structure

The existing structure of the Nursing Faculty Building, Andalas University consists of the foundation, slab and first floor columns, as shown in Fig. 1.

Fig. 1. Existing Building Condition

From a field survey on the existing structure, it is found that the steel reinforcement had corroded (Fig. 2) and the concrete has porous and overgrown with moss (Figs. 3 and 4). From the concrete quality test results using a hammer test, the average concrete quality is fc'= 11.24 MPa, where the quality of this concrete is not in accordance with the design concrete quality.

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Fig. 2. Steel Reinforcement Corrosion

Fig. 3. The Concrete Surface is Overgrown with Moss

Fig. 4. Porous Concrete and Rusty Steel Reinforcement

2.2 Structural System

The structural type of the Nursing Faculty Building, Andalas University, is a reinforced concrete structure. The building risk category is type IV because the building is classified as an educational facility building with seismic design category D, so the structural system used in the analysis is a special moment resisting reinforced concrete frame system.

2.3 Building Structure Data

Data on the structure of the Nursing Faculty Building can be seen in Table 1.

Table 1. Building data

Name Of Building	Nursing Faculty Building of Andalas University
Addres	Limau Manis, Padang
Structure Type	Concrete Reinforced
Design Concrete Quality	fc'=22,39 MPa
Eksisting Concrete Quality	fc'=11,24 MPa
Number Of Stories	2 stories with concrete slab
Building Height	8 Meter
Column Dimension	· Main Column (K1): (40x40) cm
Beam Dimension	· Main Beam (BA) : (30x50) cm
	· Secondary Beam (BA-1): (20x30) cm
Slab Thickness	12 cm
Length	2600 cm
Width	2200 cm

2.4 Modeling of Structure

Analysis of the Nursing Faculty Building structure, Andalas University, was carried out using 3D structural modeling in the ETABS program. Columns and beams of the building structure are modeled as frame elements, while the slabs are modeled as slab elements.

The results of the modeling of the building structure can be seen in Fig. 5.

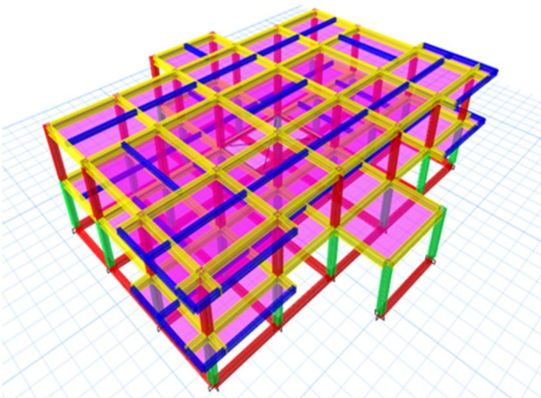


Fig. 5. Modeling of Structure

2.5 Building Loads

2.5.1 Dead Load

Based on Article 3.1.1 of SNI 1727:2020, the dead load is defined as the weight of all installed building construction materials, including walls, floors, roofs, ceilings, stairs, fixed partition walls, finishing, building cladding, and other architectural and structural components as well as service equipment. Other installed equipment includes crane weights and material conveying systems [2].

Dead loads on the Nursing Faculty Building:

a. Self-weight of structural elements is calculated directly by the ETABS structural analysis program version 2016.
b. Weight of floor covering (ceramic) 1cm thick = 1 x 24 kg/m² = 24 kg/m²
c. Ceiling weight = 20 kg/m²
d. Flooring mortar 2 cm thick floor = 2 x 21 kg/m² = 42 kg/m²
e. Mechanical Electrical and Plumbing weight = 25 kg/m²
f. Waterproofing weight= 5 kg/m²

2.5.2 Live Load

Article 4.3.1 of SNI 1727:2020 explains that the live load used in the design of buildings and other structures must be the maximum load that is expected to occur due to the occupancy and use of the building, but it must not be less than the minimum uniform load specified in Table 4.3.1 on SNI 1727:2020 [2]. The live loads acting on the building:

1. Corridor above first floor = 3.83 kN/m²
2. Office = 2.4 kN/m²
3. Meeting room = 4.79 kN/m²
4. Toilet = 2.87 kN/m²

2.5.3 Brick Wall Load

In addition to the dead load acting on the floor of the building, another load acting on the building is the brick wall. According to SNI 1727:2020, the load on the brick walls working on the building is 250 kg/m² while the wall height is 4 m, so the uniform load acting on the building with brick walls is 4m x 250 kg/m² = 1000 kg/m [2].

2.5.4 Partition Loads

Based on SNI 1727:2019 Article 4.3.2, it can be seen that the partition load is at least 72 kg/m² [2]. So that for the Nursing Faculty Building, it is assumed that the partition weight is 80 kg/m² and for the height of the building between floors is 4 meters, the partition load acting on the floor beams of the Nursing Faculty Building is 4 m x 80 kg/m² = 320 kg/m² (3.14 kN/m).

2.5.5 Earthquake Loads

Earthquake load analysis is conducted based on SNI 1726:2019 [3]. The type of earthquake load used in the design of the structure is dynamic earthquake load. The dynamic earthquake load used in building structures is the response spectrum. The earthquake spectrum response data itself is obtained from the RSA Puskim PUPR application. The Nursing Faculty Building is located in Padang city. The response spectrum data is shown in Tables 2 and 3.

Table 2. Spectra data (Source: PUSKIM PUPR RSA Application)

Variable	Value
PGA (g)	0.588
S_S (g)	1.480
S_1 (g)	0.600
C_{RS}	0.000
C_{R1}	0.000
F_{PGA}	1.000
F_A	1.000
F_V	1.700
PSA (g)	0.536
S_{MS} (g)	1.480
S_{M1} (g)	1.020
S_{DS} (g)	0.987
S_{D1} (g)	0.680
T_0	0.138
T_S	0.689

Table 3. Spectrum response period

Period (T)	Sa(g)
0.000	0.000
0.100	0.824
0.300	0.987
0.500	0.987
1.000	0.680
1.500	0.453
2.000	0.340
2.500	0.272
3.000	0.227
4.000	0.170

These tables are dynamic earthquake data obtained from the Puskim PU application. From these data, a graph of Earthquake Response Spectrum Design of Padang City was obtained, as shown in Fig. 6.

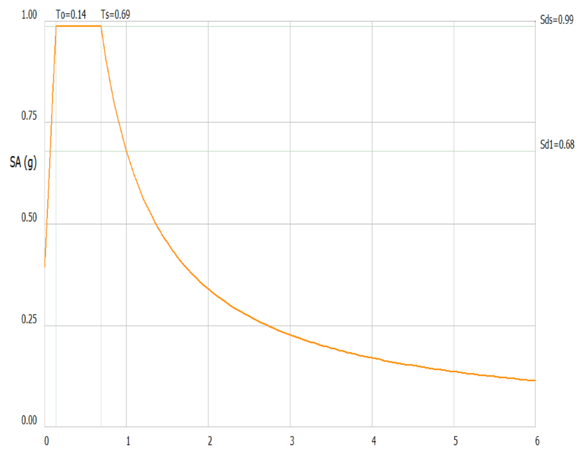


Fig. 6. Earthquake Response Spectrum Design of Padang City

2.6 Load Combination

The combination of structural loading refers to the SNI 1726:2019 [3]:

1. 1, 4D
2. 1.2 D + 1.6 L + 0.5 (Lr or R)
3. 1.2 D + 1.6 (Lr or R) + (L or 0.5 W)
4. 1.2 D + 1.0 W + L+ 0.5 (Lr or R)
5. 0.9 D+ 1.0 W
6. 1,2D + EV + EH + L
7. 0.9D - EV + EH

Table 4. Load combination

Loads Combination												
1	1.4	DL										
2	1.2	DL	+	1.6	LL							
3	1.397	DL	+	1	LL	+	1.3	EQX	+	0.39	EQY	
4	1.397	DL	+	1	LL	+	1.3	EQX	-	0.39	EQY	
5	1.397	DL	+	1	LL	-	1.3	EQX	+	0.39	EQY	
6	1.397	DL	+	1	LL	-	1.3	EQX	-	0.39	EQY	
7	1.397	DL	+	1	LL	+	0.39	EQX	+	1.3	EQY	
8	1.397	DL	+	1	LL	+	0.39	EQX	-	1.3	EQY	
9	1.397	DL	+	1	LL	-	0.39	EQX	+	1.3	EQY	
10	1.397	DL	+	1	LL	-	0.39	EQX	-	1.3	EQY	
11	0.703	DL	+	1.3	EQX	+	0.39	EQY				
12	0.703	DL	+	1.3	EQX	-	0.39	EQY				
13	0.703	DL	-	1.3	EQX	+	0.39	EQY				
14	0.703	DL	-	1.3	EQX	-	0.39	EQY				
15	0.703	DL	+	0.39	EQX	+	1.3	EQY				
16	0.703	DL	+	0.39	EQX	-	1.3	EQY				
17	0.703	DL	-	0.39	EQX	+	1.3	EQY				
18	0.703	DL	-	0.39	EQX	-	1.3	EQY				
19	Envelope											

Table 4 shows the load combinations in the structural analysis of the Nursing Faculty Building.

2.7 Inter Story Drift

Table 5. Inter story drift limit (Source: SNI 1726:2019)

Structure	Risk Category		
	I or II	III	IV
Structures, other than brick shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall system which has been designed for accommodate deviations between levels.	0,025hsx	0,020hsx	0,015hsx
Brick cantilever shear wall structure	0,010 hsx	0,010 hsx	0,010 hsx
Other brick shear wall structures	0,007 hsx	0,007 hsx	0,007 hsx
All other structures	0,020 hsx	0,015 hsx	0,010 hsx

Table 5 shows the boundary conditions for the inter story drift for the building. From the equation in the table, the calculation results of the building inter story drift are shown in Tables 6 and 7.

Table 6. Inter story drift X direction

Story	Direction	δ_e	Cd	δ_{ex}	Δ_x	Height	Δ(Limit)	Description
		mm		mm	mm	mm	mm	
2	X	31.393	5.5	115.10767	52.7817	4000	40	NOT OK
1	X	16.998	5.5	62.326	62.326	4000	40	NOT OK
0	X	0	5.5	0	0	4000	40	OK

Table 7. Inter story drift Y direction

Story	Direction	δ_e	Cd	δ_{ex}	Δ_x	Height	Δ(Limit)	Description
		mm		mm	mm	mm	mm	
2	Y	30.403	5.5	111.4777	51.5167	4000	40	NOT OK
1	Y	16.353	5.5	59.961	59.961	4000	40	NOT OK
0	Y	0	5.5	0	0	4000	40	OK

From Tables 6 and 7, it can be seen that the inter story drift that occurs in the building does not meet the permit limit requirements in SNI 1726:2019.

2.8 Cross-sectional Capacity of the Structure

2.8.1 Column Capacity

The capacity of the column is determined through the interaction diagram of the axial moment and compression of the column and the shear capacity of the column [4].

a. Momen and Axial of Column

The calculation results of the interaction diagram of the first floor and second floor columns of the building are shown in Figs. 7 and 8.

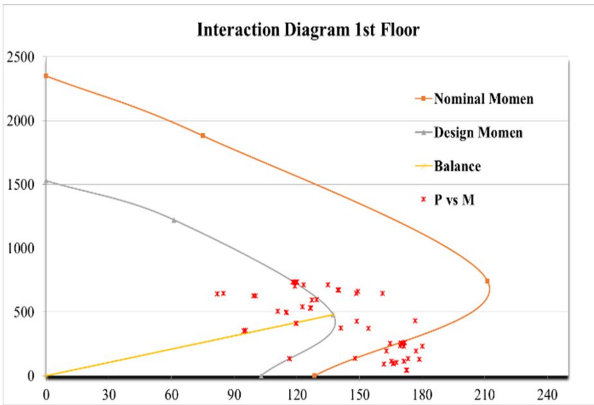


Fig. 7. First Floor Column P-M Interaction Diagram

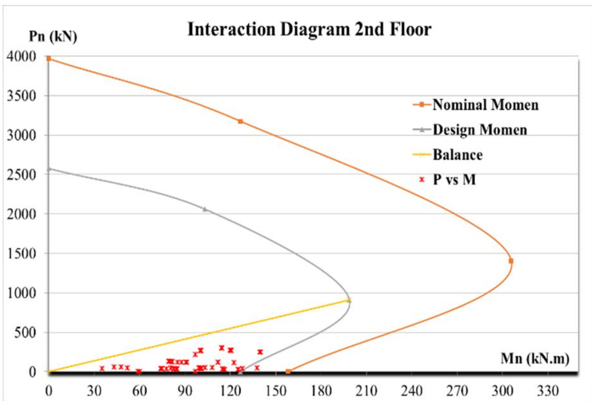


Fig. 8. Second Floor Column P-M Interaction Diagram

As seen in Figs. 7 and 8, the first and second floor columns are not strong enough to carry the working load because some axial moment and compression forces pass through the design axial compression moment line.

b. Shear Capacity of Columns

The shear capacity of the building columns is shown in Table 8. From the table, it is clearly seen that the column is able to withstand the shear forces acting on the structure.

Table 8. Shear capacity column

	Dimension (mm)	Fy	Dim. Shear Reinforced	Spasi	ΦVn	Vu	Description
1	400 x 400	310	10	100	2250.719	101.1666	OK
2	400 x 400	400	10	100	2542.005	138.0258	OK

2.8.2 Beam Capacity

a. Flexural Capacity of Beams

Table 9 shows the flexural capacity of the building beams.

Table 9. Beam flexure capacity

Type (Dimension)	Location	Main Reinforced		Nominal Flexural Capacity		Ultimate Flexural Forces		Description
		At Support	At Field	At Support	At Field	At Support	At Field	
Sloof (30X50)	Top	4D16	4D16	116.89	116.89	60.07	15.02	OK
	Bottom	4D16	4D16	116.89	116.89	30.03	30.03	
1st floor Beam (30X50)	Top	6D16	4D16	181.37	125.00	228.83	64.17	NOT OK
	Bottom	4D16	6D16	125.00	181.37	162.12	37.57	
2nd floor Beam (30X50)	Top	6D16	4D16	181.37	125.00	150.30	37.57	OK
	Bottom	4D16	6D16	125.00	181.37	93.85	82.35	
1st floor Ba-1 (20x30)	Top	5D16	3D16	74.92	49.56	63.69	15.92	NOT OK
	Bottom	3D16	5D16	49.56	74.92	81.70	98.43	
2st floor Ba-1 (20x30)	Top	5D16	3D16	74.92	49.56	64.59	16.15	NOT OK
	Bottom	3D16	5D16	49.56	74.92	44.57	85.32	

From Table 9, it can be seen that the beams on the first floor and the beams on the first and second floors are unable to withstand the loads acting on the structure.

b. Shear Capacity of Beams

The shear capacity of the building beams can be seen in Table 10. From the table, it is found that the beam is able to withstand the shear forces acting on the structure.

Table 10. Beam shear capacity

Type (Dimension)	Shear Reinforced		Nominal Shear Capacity		Ultimate Shear Forces		Description
	At Support	At Field	At Support	At Field	At Support	At Field	
Sloof (30X50)	Ø10-100	Ø10-150	366.14	269.79	25.65	55.02	OK
1st floor Beam (30X50)	Ø10-100	Ø10-150	397.86	301.52	243.54	219.82	OK
2nd floor Beam (30X50)	Ø10-100	Ø10-100	397.86	301.52	143.20	117.88	OK
1st floor Ba-1 (20x30)	Ø12-100	Ø10-150	276.25	149.92	82.27	50.23	OK
2nd floor Ba-1 (20x30)	Ø12-100	Ø10-150	276.25	149.92	71.20	52.08	OK

Based on the examination of the mass participation factor, scale factor, P-delta, and structural irregularities, all of them have met the requirements of SNI 1726:2019 [3].

From the evaluation results of the structural building performance, it was found that the structure of the columns and beams could not resist the loads acting on the structure. In addition, the inter story drift that occurred did not meet the permit limit requirements according to Indonesian building standards. Therefore, the building should be retrofitted before continuing the construction.

3 Retrofitting of the Structure

Analysis of the retrofitting/strengthening of the structure in this building is by re-design the Detail Engineering Design (DED) of the structure where the existing columns are jacketed [5-7]. The re-design is carried out on the beams, the second-floor column, while the first-floor column is retrofitted using the jacketing method.

3.1 Beam

Re-design of beams was carried out on the structure of the main and secondary beams of the building. The re-design was carried out because the beams from the initial design (DED) could not withstand the loads acting on the structure [4,8].

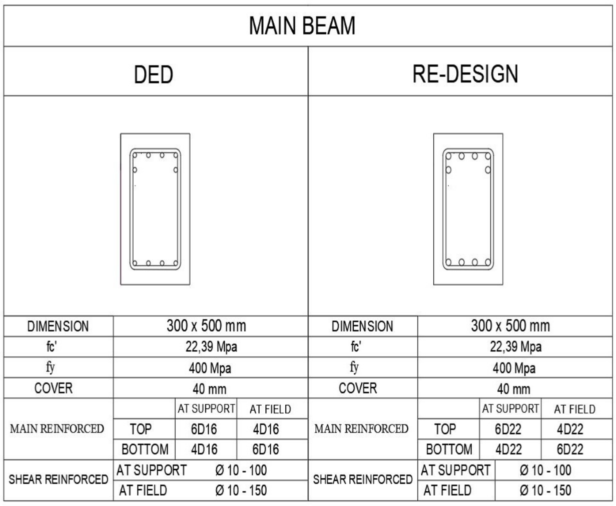


Fig. 9. Comparison of the Main Beam Details between the DED (Initial Design) and the Re-design

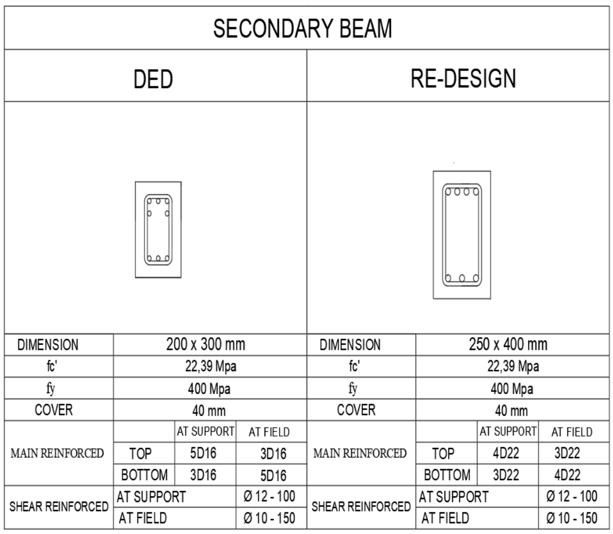


Fig. 10. Comparison of the Secondary Beam Details between the DED (Initial Design) and the Re-design

Figs. 9 and 10 show the comparison of the details of the main beam between the initial design (DED) and the Re-design.

3.1.1. Re-designed Beam Capacity

a. Flexural Capacity

The results of the beam flexural capacity analysis are shown in Table 11.

Table 11. Re-designed beam flexural capacity

Type (Dimension)	Location	Main Reinforced		Nominal Flexural Capacity		Ultimate Flexural Forces		Description
		At Support	At Field	At Support	At Field	At Support	At Field	
Sloof (30X50)	Top	4D16	4D16	116.89	116.89	60.07	15.02	OK
	Bottom	4D16	4D16	116.89	116.89	30.03	30.03	
1st floor Beam (30X50)	Top	6D22	4D22	311.90	222.56	178.07	44.52	OK
	Bottom	4D22	6D22	222.56	311.90	98.20	137.82	
2nd floor Beam (30X50)	Top	6D22	4D22	311.90	222.56	149.82	37.46	OK
	Bottom	4D22	6D22	222.56	311.90	97.08	81.66	
1st floor Ba-1 (20x30)	Top	4D22	3D22	161.97	128.06	73.67	24.92	OK
	Bottom	3D22	4D22	128.06	161.97	84.52	158.07	
2st floor Ba-1 (20x30)	Top	4D22	3D22	161.97	128.06	72.56	18.14	OK
	Bottom	3D22	4D22	128.06	161.97	47.85	91.23	

From Table 11, it can be seen that all the beam structures after the re-design have been able to withstand the loads acting on the structure.

b. Shear Capacity

Table 12 shows the results of the beam shear capacity analysis.

Table 12. Re-design beam shear capacity

Tipe (Dimensi)	Shear Reinforced		Nominal Shear Capacity		Ultimate Shear Forces		Description
	At Support	At Field	At Support	At Field	At Support	At Field	
Sloof (30X50)	Ø10-100	Ø10-150	366.14	269.79	25.65	55.02	OK
1st floor Beam (30X50)	Ø10-100	Ø10-150	397.86	301.52	240.66	190.41	OK
2nd floor Beam (30X50)	Ø10-100	Ø10-100	397.86	301.52	146.26	123.36	OK
1st floor Ba-1 (20x30)	Ø12-100	Ø10-150	396.70	221.77	93.19	34.88	OK
2nd floor Ba-1 (20x30)	Ø12-100	Ø10-150	396.70	221.77	51.36	48.95	OK

From Table 12, it is found that the re-designed beam is able to withstand shear forces due to external loads on the structure.

3.2 Second Floor Column

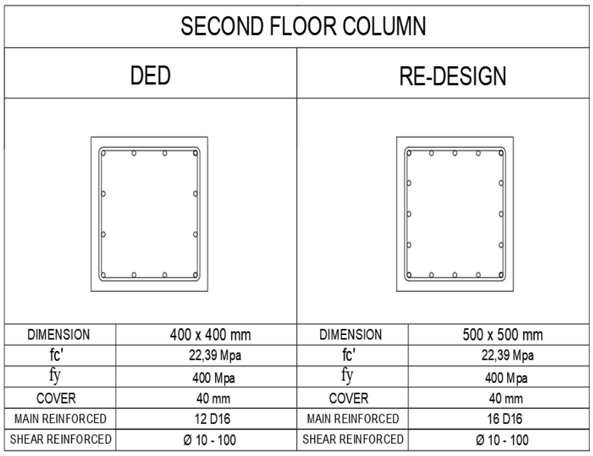


Fig. 11. Initial design (DED) and Re-design Column Details

The re-design of columns was carried out on the second floor columns of the building because the capacity of the planned columns could not withstand the working loads, so the dimensions and reinforcement of the column had to be changed [4,8].

A detailed comparison between the initial design (DED) and re-designed second floor columns can be seen in Fig. 11.

3.2.1. Column Capacity

Fig. 12 shows the second floor column P-M interaction diagram.

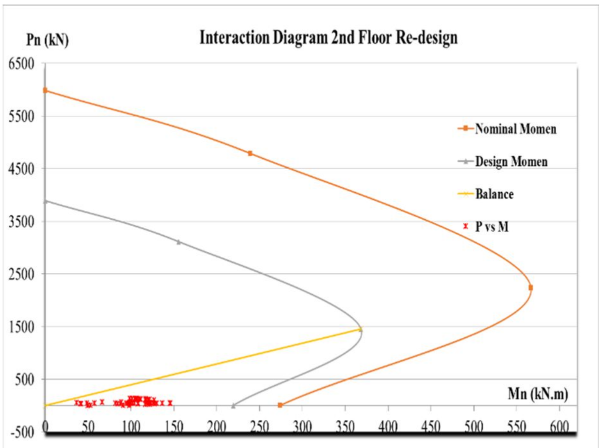


Fig. 12. Second Floor Column P-M Interaction Diagram Re-design

As seen in Fig. 12, all P-M values on the graph have been in the interaction diagram line indicating that all building columns are capable of carrying the working load.

3.3 Retrofitting of First Floor Column using Jacketing Method

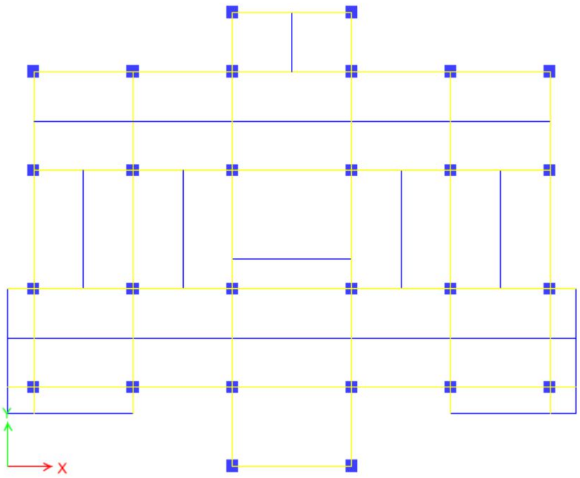


Fig. 13. Position of the Concrete Jacketing Column

Retrofit of the first floor columns is carried out using the Concrete Jacketing method on all existing columns because the concrete and steel quality in the column has decreased [9,10]. The Concrete Jacketing Column Position is shown in Fig. 13.

3.3.1. Structural Retrofitting Modeling using Concrete Jacketing Method

Retrofitting of the column using jacketing method is carried out by increasing the dimensions and adding steel reinforcement to the column with the following assumptions

- The planned jacketing column is 600 x 600 (mm).
- The column quality to be achieved is fc'=25 MPa.
- In order to reach a concrete quality of fc'=25 MPa, the quality of the jacketing concrete used as an addition to the column dimensions is a minimum of fc'= 36 MPa. Calculation of the quality of the concrete jacketing is as follows:
  - Column quality to be achieved (A)= 25 MPa
  - Existing Concrete Quality (B) = 11.24 MPa
  - Planned column area (C) = 600 x 600 = 360,000 mm²
  - Existing column area (D) = 400 x 400 = 160,000 mm²
  - Jacketing area (E) = 200,000 mm²

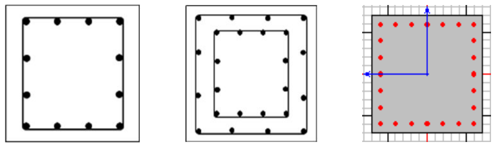
  Quality of concrete jacketing:

  $$(D \times B) + (E \times X) = C \times A$$

  $$(160{,}000 \times 11.24) + 200{,}000 \times (X) = 360{,}000 \times 25$$

  $$X = 36 \text{ MPa}$$
- The quality and amount of added steel reinforcement are the same as the existing one, that is fy= 350 MPa with 12 D16.

From Figs. 14 and 15, it can be seen the assumption of the definition of the jacketing column in the ETABS software. Fig. 16 shows the comparison of the column cross-section between the existing column and the jacketing column.



(a) Existing Column

(b) *Concrete Jacketing Coluumn*

(c) Jacketing Column with Evenly Distributed Reinforcement

Fig. 14. Conrete Jacketing Column Reinforcement Modeling

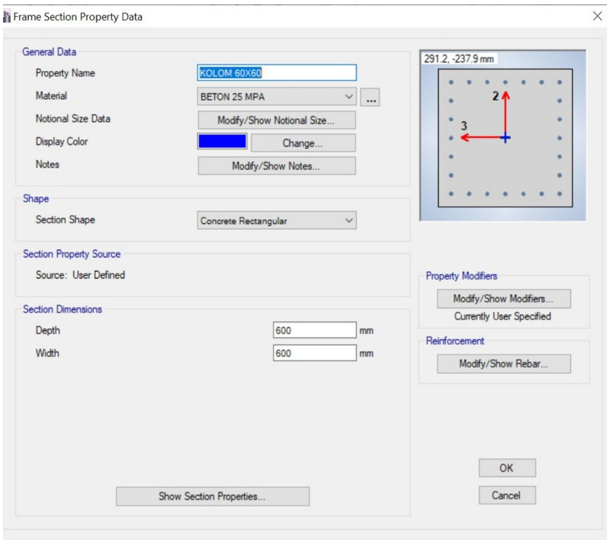
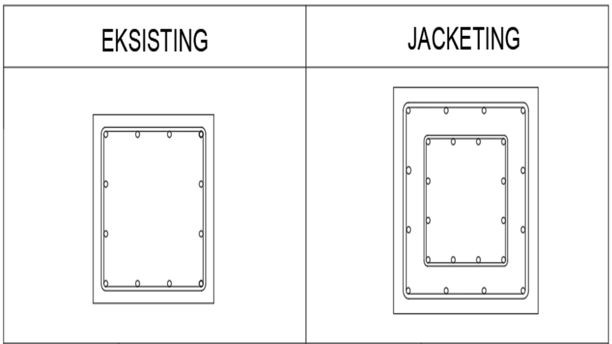


Fig. 15. Concrete Jacketed Column Modeling



DIMENSION	400 x 400 mm	DIMENSION	600 x 600 mm
fc'	22,39 Mpa	fc'	25 Mpa
fy	350 Mpa	fy	400 Mpa
COVER	40 mm	COVER	40 mm
MAIN REINFORCED	12 D16	MAIN REINFORCED	24 D16
SHEAR REINFORCED	Ø 10 - 100	SHEAR REINFORCED	Ø 10 - 100

Fig. 16. The comparison of the Column Cross-section between the Existing Column and the Jacketed Column

3.3.2. Retrofitted Column Capacity

Fig. 17 shows the cross-sectional capacity of the column reinforced by the jacketing method. From the figure, it can be seen that all the P-M points on the graph have been in the interaction diagram, so that the column is able to withstand the working load.

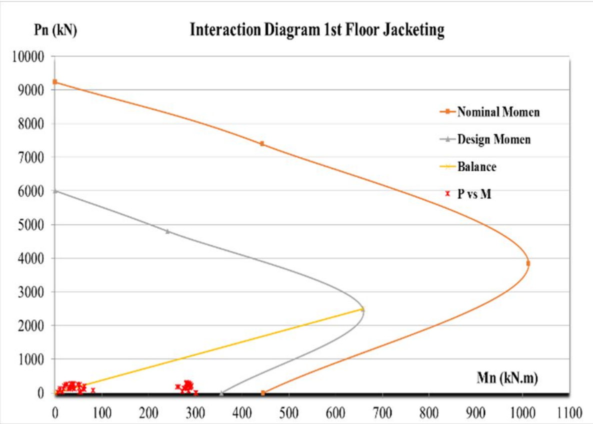


Fig. 17. P-M Interaction Diagram of Jacketed Column 600x600

3.3.3. Inter Story Drift

Retrofitting on the existing first floor column with the jacketing method also affects the building displacement, so it is necessary to check the displacement of the building structure. The calculation results of the inter story drift in the X and Y directions on the retrofitted building can be seen in Tables 13 and 14.

From Tables 13 and 14, it is clearly seen that the inter story drift that occurs have met the permit limit requirements.

Table 13. Inter story drift in X direction for retrofitted column

Story	Direction	δ_e	Cd	δ_{ex}	Δ_x	Height	Δ(Limit)	Description
		mm		mm	mm	mm	mm	
2	X	16.244	5.5	59.561	36.971	4000	40	OK
1	X	6.161	5.5	22.590	22.590	4000	40	OK
0	X	0	5.5	0.000	0.000	4000	40	OK

Table 14. Inter Story drift in Y direction for retrofitted column

Story	Direction	δ_e	Cd	δ_{ex}	Δ_x	Height	Δ(Limit)	Descripti
		mm		mm	mm	mm	mm	on
2	Y	16.131	5.5	59.147	36.938	4000	40	OK
1	Y	6.057	5.5	22.209	22.209	4000	40	OK
0	Y	0	5.5	0	0.000	4000	40	OK

4 Conclusion

Based on the structural evaluation analysis conducted at the Nursing Faculty Building, the following conclusions can be drawn:

1. The quality of the existing concrete obtained from the hammer test results is fc'=11.24 MPa, where this value is far below the concrete quality standard for reinforced concrete building structures (minimum fc' = 17 MPa).
2. The structure of the existing first floor column, second floor plan column, and the initial design of the main and secondary beams are not able to withstand the working load.
3. The inter story drift in the existing building does not meet the permit limit requirements according to the Indonesian building standard.
4. Retrofitting of the building structure is designed by re-designing the Detail Engineering Design (DED) structure, where the existing first floor column is retrofitted by using jacketing method.
5. The retrofitted building structure has a strong enough capacity to carry the working load, and the inter story drift has met the permit limit requirements.

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