ISSN (e): 2250 – 3005 || Volume, 15 ||Issue, 12||December-2025 ||
International Journal of Computational Engineering Research (IJCER)

Substandard Concrete Quality Analysis and Retrofit Feasibility in Post-Damage Two-Story Buildings

[1]Septindra Hafizh Al. Furqan
Universitas Negeri Semarang,
Sekaran, Kec. Gn. Pati, Kota Semarang, Jawa Tengah 50229 Indonesia

ABSTRACT
This study analyzes the structural condition of the Surakarta City DPRD Building after the damage using a combination of visual inspection, non-destructive testing (Hammer Test and PUNDIT), and numerical analysis based on ETABS and ABAQUS software. The results of the Hammer Test show that the quality of the concrete is in the range of 16–19 MPa and is below the minimum standard of K-250 according to SNI 2847:2019. PUNDIT testing confirms the inhomogeneity of concrete quality, where block elements have a better material density than columns and floor plates. ETABS analysis showed weaknesses in the lateral rigidity system, overstress in more than 68% of column elements, and non-conformance of column dimensions to the minimum standard. Further analysis using the Finite Element Method (FEM) showed that the existing columns work close to the limit of the material's capacity, while the jacketing simulation only provides a limited and potentially ineffective strength increase due to degradation of the old concrete and the risk of slip in the contact plane. Based on the integrative results of all stages of analysis, the building structure is considered unsuitable for maintenance and total reconstruction action is recommended as the safest technical solution to ensure structural performance in accordance with the provisions of SNI 2847:2019 and SNI 1726:2019.
KEYWORDS: *Substandard concrete; Hammer Test; PUNDIT; ETABS; ABAQUS; FEM; lateral stiffness; overstress; jacketing; structural evaluation; SNI 2847:2019; SNI 1726:2019.*

Date of Submission: 03-12-2025 Date of acceptance: 15-12-2025

I. INTRODUCTION

The Surakarta City Regional People's Representative Council (DPRD) Building is a government facility that plays a vital role as a center for legislative activities and public services. As the main infrastructure of the government, the sustainability of building functions is highly dependent on the safety and feasibility of the building structure. This is in line with Rohman's (2025) view that government buildings must have high structural safety standards to maintain the sustainability of public services.

Damage to structural elements that arise after social events causes degradation of concrete strength, structural cracks, and a decrease in the rigidity of elements, so it is necessary to re-evaluate the building structure. Wongso Suharto and Fuk Jin (2024) stated that demolition and reconstruction can be the right choice if the structure no longer meets the minimum strength requirements.

The initial evaluation was carried out through visual inspection, followed by a hammer test and PUNDIT as a non-destructive test (NDT) to assess the quality of the concrete in-situ. This method is widely used in research, and Herdiansah & Cendana (2025) proved that there is a strong correlation between the value of hammer rebound and ultrasonic pulse velocity (UPV) on the actual quality of concrete. Ridho & Khoeri (2024) research also confirms that the combination of the two methods is effective in determining the degradation of concrete in old buildings.

In addition, Faisal Ridho (2015) stated that the UPV test can detect internal cracks and inhomogeneity of concrete that cannot be seen visually. In line with that, Malhotra & Carino (2004) as a classic reference in NDT also emphasize that ultrasonic wave-based methods and energy reflection are reliable indicators for detecting structural damage.

The field data was then modeled in the ETAB software, referring to the provisions of SNI 1726:2019 (earthquake resistance), SNI 1727:2020 (building loading), and SNI 2847:2019 (reinforced concrete structure). This analysis is necessary to identify elements that are overstressed due to gravitational and lateral loads. According to Chopra (2017), the analysis of the structural response to earthquakes should include the evaluation of shape modes, dynamic responses, and deformation between floors to ensure that the structure has adequate ductility and energy capacity.

The modeling results showed that there were several elements that exceeded the permit limit, so further analysis was carried out using the Finite Element Method (FEM) through ABAQUS. The FEM approach is highly effective in mapping stress distribution, local collapse, and plastic deformation. Zhang et al. (2025) explained that FEM is able to provide a detailed picture of voltage concentration, especially in joints, stacks, and areas that experience significant cracking.

In the context of the redesign of public buildings, Fauzan et al. (2024) highlight that retrofit efforts are often ineffective on structures with high levels of damage, as old concrete tends to be incompatible with new elements, thus triggering stress concentrations and the risk of long-term fatigue. Therefore, under certain conditions, the act of total dismantling and rebuilding is more recommended than partial reinforcement.

In addition to structural factors, this study considers aspects of disaster preparedness and public safety. Mahmoudi et al. (2026) show that the application of performance-based seismic design can increase the resilience of public buildings to large earthquakes. Similarly, Nastri (2025) emphasized the importance of evaluating the behavior of structures at various levels of earthquake intensity to ensure that buildings remain functional after the event.

The earthquake that occurred in the Java region, including Surakarta, further strengthened the urgency of planning earthquake-resistant structures. Goro et al. (2024) noted that the Java region has a medium to high level of earthquake danger based on the acceleration of the ground peak (PGA). Meanwhile, the Opak, Baribis, and Lasem faults are potential sources of earthquakes that can affect Surakarta, so building planning such as the DPRD Building must comply with all the provisions of the latest SNI earthquake.

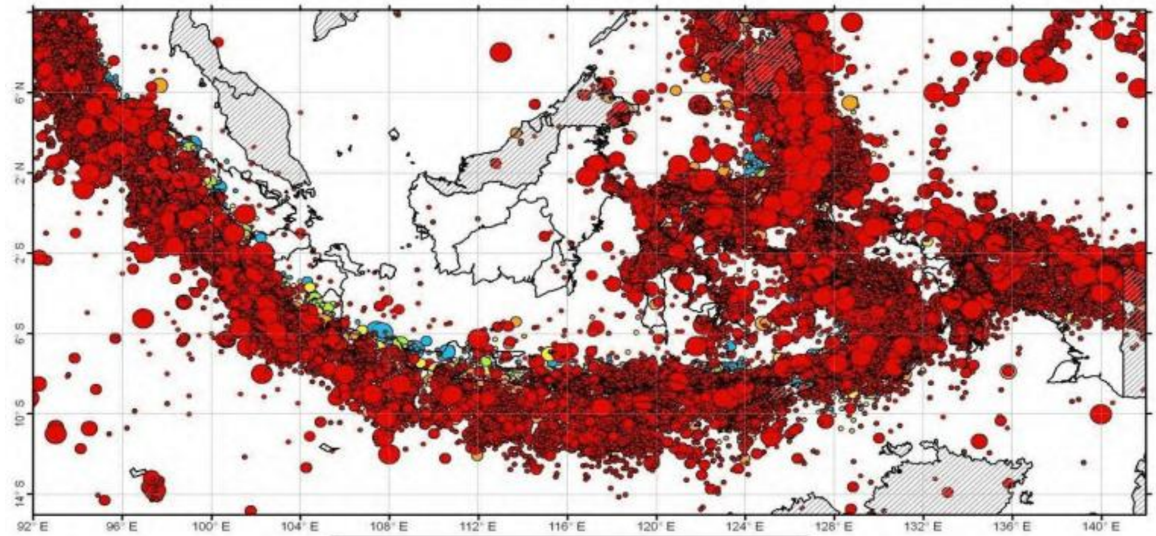


Figure 1. Map of earthquake distribution in Indonesia

Taking into account the results of visual inspections, NDT tests, ETAB modeling, and FEM ABAQUS analysis, this study confirms that most of the structural elements of the Surakarta City DPRD Building are no longer suitable for maintenance. The main recommendation is to carry out the demolition and replanning of the structure by adopting the principles of resilient design, material efficiency, and the fulfillment of all SNI requirements. This reconstruction effort is expected to be able to produce safer, resilient, and sustainable government buildings, according to the needs of modern public services (Wongso Suharto & Fuk Jin, 2024).

II. METHOD

The methodology of this study is designed to provide a comprehensive assessment of the structural condition of the Surakarta City Regional People's Representative Council (DPRD) Building through empirical approaches, field experiments, and computer-based numerical analysis. This method aims to produce a scientifically accountable evaluation of the integrity of structural elements and determine the feasibility of buildings to be maintained, strengthened, or completely reconstructed.

Field Data Collection

The first stage in the methodology is the primary data collection activity through a thorough visual inspection of all structural elements. Observations are made on column components, beams, plates, and joints to identify indications of damage including:

i. retak struktural (structural cracks),
ii. retak rambut (hairline cracks),
iii. spalling concrete,
iv. korosi tulangan (reinforcement corrosion),
v. vertical and lateral deformation,
vi. indication of a decrease in the stiffness of the elements.

Visual inspections are carried out systematically to map the extent of damage to elements and determine areas that require further testing. This observation is the foundation in understanding the phenomenon of initial damage that can affect the structural capacity of the building as a whole.

Non-Destructive Testing (NDT)

Non-destructive testing is used to obtain material parameters without damaging structural elements. Two types of tests are used as a concrete quality verification tool in situ, namely:

i. Hammer Test (Schmidt Hammer Test)

Hammer test is applied to estimate the compressive strength of concrete based on the rebound value of the concrete surface. This method refers to SNI ASTM C805:2012. The stages of implementation include:

- The placement of the tool perpendicular to the concrete surface so that the impact occurs optimally.
- Controlled impact delivery using the hammer's internal energy.
- Recording of the reflected numbers read on the tool.
- Repeat the test for at least 10 points in each area to obtain a representative score.
- Evaluate the test surface to ensure no local damage due to cavities or inhomogeneity.

This method provides a preliminary picture of the uniformity and hardness of the concrete surface, which is correlated with the approximate value of f'c.

Figure 2. Hammer Test Matest type 380

ii. Pulse Ultrasonic Non-Destructive Test (PUNDIT)

PUNDIT is used to assess concrete density, material homogeneity, and detect the presence of internal cracks through ultrasonic wave propagation speed measurement. The testing stages include:

- Calibration of the tool using standard media to ensure accuracy.
- Surface preparation through smoothing (if required) for optimal wave transmission.
- Couplant gel application to avoid air traps on sensor surfaces.
- The selection of test modes (direct, semi-direct, surface) is based on field conditions.
- Ultrasonic pulse velocity readings are then analyzed against possible internal damage.

Low propagation speed indicates high porosity, cracking, or deterioration in the quality of concrete. This data is then combined with the results of the hammer test for a more accurate assessment of the material.

Figure 3. PUNDIT Test Kit

Technical Data Processing

The results of field tests are processed into material parameters used in numerical modeling. The transformed variables include:

i. in-situ concrete compressive strength (f'c) value,
ii. ultrasonic wave propagation speed,
iii. Internal Integrity Level.
iv. value of material homogeneity.

Data processing is carried out to ensure that the structure model represents the actual condition of the building realistically.

Structural Modeling and Analysis Using ETABS

Numerical analysis was performed with ETABS to evaluate the global behavior of the structure. The structural model is developed based on:

i. building geometry,
ii. the dimensions of each element,
iii. The actual material of the NDT,
iv. the structure of the moment bearer skeleton,
v. planning regulations SNI 1726:2019 and SNI 1727:2020.

The analysis carried out includes:

a. Gravity and Lateral Load Analysis
The model is analyzed against the combination of loads:
- dead load (D),
- live load (L),
- Wind load (W),
- earthquake load (E) based on the response value of the spectrum of the Surakarta area.

b. Spectrum Response Analysis
Spectrum response is used to understand the dynamic behavior of structures to earthquake intensity at various vibration frequencies. The analyzed parameters include:
- fundamental period of the building,
- Mode Shape,
- Modal Participation Mass Ratio,
- Inter-story drift.

c. Evaluation of Element Capacity
Each structural element—columns, beams, and plates—is checked for:
- flexural capacity,
- shear capacity,
- deflection control,
- overstress,
- ductility and seismic performance requirements.

Detailed Analysis of Structural Elements Using ABAQUS (FEM)

Elements that show indications of failure or are overloaded beyond the plan's capacity are analyzed locally using the Finite Element Method (FEM) via ABAQUS. This analysis is important to understand the behavior of elements in a micro-way through:

i. mesh modeling and solid elements,
ii. nonlinier material analysis,
iii. stress distribution,
iv. distribusi regangan (strain distribution),
v. identification of plastic zones,
vi. predicting local failure,
vii. post-degradation capacity evaluation.

The FEM method provides a very detailed representation so that it is able to reveal the phenomenon of failure that cannot be seen in conventional structural analysis.

Evaluation of Structural Feasibility

All observations, material tests, and numerical analysis were then evaluated against the provisions of structural feasibility based on SNI. An assessment is carried out to determine:

i. whether the structural elements still meet the plan's capacity,
ii. whether retrofitting can be done, or
iii. Whether a total demolition (rebuild) action is required.

The evaluation considers the following aspects:

i. structural strength,
ii. material durability,
iii. global stability,
iv. boundary boundaries,
v. Safety factors against earthquakes,
vi. risk of progressive failure.

Preparation of Structural Engineering Recommendations

The final stage of this methodology is the preparation of technical recommendations based on the overall results of the analysis, which includes:

i. recommendations for demolition or repair,
ii. selection of new structural systems that are more earthquake-resistant,
iii. adjustment of design load according to the latest standards,
iv. the application of performance-based design principles,
v. improved material efficiency and structural sustainability.

III. RESULTS AND DISCUSSION

Data and Analysis of Hammer Test Results

Based on the results of the calculation analysis of the hammer test data that has been taken in the field, the estimated value of the compressive strength of concrete for all points taken in Table 1 is obtained as follows (for the complete results of the hammer test can be seen in the appendix).

Table 1. Recap of Hammer Test Results

Location	Kode	Notation in the Plan	Hammer Test Results	
			Concrete Compressive Strength Value (kg/cm2)	F'c (Mpa)
1st Floor Column Structure	K1 B5	1	190	18.6
	K1 C4	2	220	21.6
	K1 D5	3	220	21.6
	K1 B4	4	220	21.6
	K1 E7	5	220	21.6
2nd Floor Column Structure	K1 C3 lt.2	7	220	21.6
	K1 E3 lt.2	8	220	21.6

Location	Kode	Notation in the Plan	Hammer Test Results	
			Concrete Compressive Strength Value (kg/cm2)	F'c (Mpa)
	K1 B4 lt.2	9	220	21.6
Beam Structure	BI D67	1	280	27.5
	BI C45	3	245	24.0
	BI CD7	4	310	30.4
	BI D45	5	310	30.4
	BI B45	6	400	39.2
	BI E57	7	370	36.3
	BA CD4	8	220	21.6
Stair StructurePlate Structure	Ladder	2	250	24.5
Struktur Pelat	P DE45	1	425	41.7
	P BC45	2	425	41.7
	P DE67	3	410	40.2
	P CD23	4	170	16.7
	P CD34	5	170	16.7

Based on the results of the Hammer Test on the structure of the Surakarta City Parliament Building, the average compressive strength of concrete was ±16 MPa with a range of 13.5–18.7 MPa. This value shows that the quality of concrete is in the range of K-175–K-200, lower than the minimum standard of K-250 for government buildings according to SNI 2847:2019. The variation in results between points indicates inhomogeneity in the quality of concrete due to material degradation and differences in the quality of construction implementation. These findings show that existing concrete does not meet the structural strength requirements, so further evaluation is needed through the PUNDIT test and numerical analysis based on ETABS and ABAQUS to determine the capacity and performance of the structure more accurately.

Data and Analysis of PUNDIT Test Results

Based on the results of the analysis from the Portable Ultrasonic Non-destructive Digital Indicator Tester (PUNDIT) test conducted in the field. The results are obtained in Table 4.3 below.

Table 2. PUNDIT Test Results

No.	Location	Code	Notation in the Plan	Compressive Strength Value Concrete
				Fc (Mpa)
1.	1st Floor Column Structure	K D5 (Horizontal)	c	16
2.	2nd Floor Column Structure	K C3 (Horizontal	E	16
3.	Master Beam Structure	B C45 (Vertikal)	A	33
4	Master Beam Structure	B CD7 (Horizontal)	B	33
5	Structure 2nd floor slab	P DE45	D	19

The results of PUNDIT's testing on the structure of the Surakarta City DPRD Building show that the beams have good quality and density of concrete, while columns and floor plates are in the medium to low category. These findings are consistent with the results of the Hammer Test, where the 1–2nd floor columns have a compressive strength of about 16 MPa, the 2nd floor plate of 19 MPa, and the main beam reaches 33 MPa. The difference in values between the elements indicates the inhomogeneity of the quality of concrete due to variations in casting methods or material degradation. Thus, some elements—especially columns and plates—show a decrease in quality, requiring further evaluation through ETABS and ABAQUS analysis to assess the actual capacity of the structure to gravitational loads and earthquakes in accordance with SNI 2847:2019 and SNI 1726:2019.

Pengumpulan Data Teknis

The structural study on the Surakarta City Regional People's Representative Council (DPRD) Building project was carried out with reference to relevant technical data to ensure accuracy in the structural analysis and modeling process. All structural planning and evaluation are carried out based on the existing condition of the building, the results of field inspections, and planning parameters in accordance with the provisions of national standards. The building is designed with technical specifications that consider its function as a government facility, public safety factors, and resistance to gravitational loads, live loads, and lateral loads (earthquakes and winds), in accordance with the provisions of SNI 1726:2019, SNI 1727:2020, and SNI 2847:2019.

- Number of Floors : 2 floors
- Total Building Area : 296.6 m2
- Building Height : 13 meters
- Structure Material : Concrete fc 25 Mpa and Steel quality fy 550 Mpa

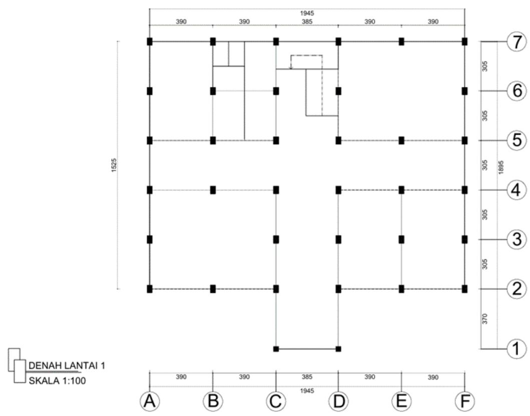


Figure 4. 1st Floor Plan

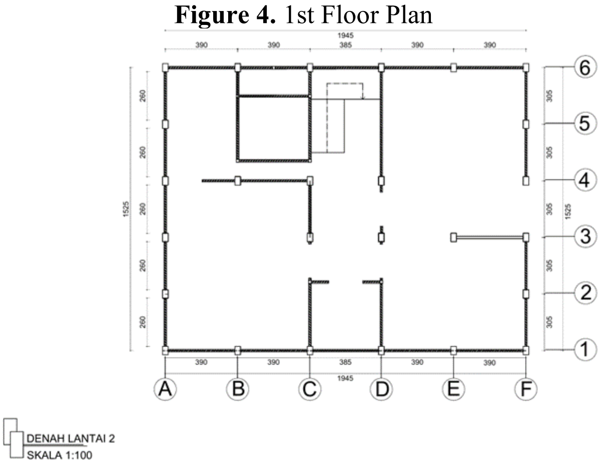


Figure 5. 2nd Floor Plan

Spesifikasi Material

The specifications of concrete and iron materials are determined as follows:

- Concrete Quality : 1). Column = fc 16 Mpa
  2). Balok = fc 19 Mpa
  3). Pelat = fc 19 Mpa

- Iron Profile Quality : BJ-55; fy=550 Mpa, Fu=620 Mpa
- Mutu baut : A-325 ; Fy=580 Mpa, Fu=875 Mpa
- Weld quality : A5.1 electrode (E 7016 or E7018 or equivalent)

Live Load Parameters

The following is the burden of life that works in the structure of the Surakarta City DPRD in accordance with SNI 1727:2020. Table 4.3 -1 Evenly distributed live load minimum Life Load (LL)

- Roof live load = 0.96 kN/m2
- R. Light Warehouse = 6.0 kN/m2
- Meeting Room = 2.87 kN/m2
- Office Space = 2.4 kN/m2
- Bathroom (toilet) = 4.0 kN/m2
- 1st floor corridor & stairs = 4.79 kN/m2
- 2nd floor corridor = 3.83 kN/m2

Additional Load Parameters

The magnitude of the loads acting due to gravitational loads can be seen in the load plan as follows:

Pembebanan Super Dead Load (SDL)

- Ceramic Load = 0.24 kN/ m2
- Light Brick Wall Load = 600kg/m3 x 4 m x 0.1 m = 240 kg/m = 2.35 kN/m
- Clay Tile Load = 45 Kg/m2 = 0.44145 kN/m2
- Light Steel Frame = 0.10 kN/m2
- Beban ME = 20 kg/m2 = 0.2 kN/m2
- Ceiling load = 18 kg/m2 = 0.18 kN/m2

Mode Shape dan Modal Participation Mass Ratio

The following results obtained from the simulations that have been carried out on the ETABS software are shown in Table 3

Table 3. Existing Shape Mode

Mode Shape	**Second**	**Translation/ Rotation**	**Cheque**
Mode 1: *Translation* X direction	0.310	Translation X Direction	Safe
Mode 2: Rotation *direction Z*	0.235	Rotation Direction Z	Not Safe
Mode 3: Translation in the Y direction	0.200	Translation Direction Y	Safe
Mass Modal Participation Ratio >90%			

Tabel 4. Modal Load Participation Ratios

Modal Load Participation Ratios				
Case	***Item Type***	***Item***	***Static***	***Dynamic***
			%	%
Modal	*Acceleration*	UX	100	100
Modal	*Acceleration*	UY	100	100
Modal	*Acceleration*	UZ	0	0

Tabel 5. Modal Load Particaption Mass Ratio

Modal Participating Mass Ratios															
Case	**Mode**	**Period**	**UX**	**UY**	**UZ**	**SumUX**	**SumUY**	**SumUZ**	**RX**	**RY**	**RZ**	**SumRX**	**SumRY**	**SumRZ**
		sec												
Modal	1	0.313	0.7961	0.0001	0	0.7961	0.0001	0	0.0001	0.7847	0.125	0.0001	0.7847	0.125
Modal	2	0.235	0.0889	0.001	0	0.885	0.0011	0	0.0014	0.1504	0.7369	0.0014	0.9351	0.862
Modal	3	0.200	0.0001	0.8572	0	0.8851	0.8583	0	0.9135	0.0006	0.0003	0.915	0.9358	0.8623
Modal	4	0.194	0.0006	0.0102	0	0.8857	0.8685	0	0.0106	0.0004	0.0041	0.9256	0.9361	0.8664
Modal	5	0.185	0.0422	0.0007	0	0.9279	0.8691	0	0.0007	0.021	7.437E-06	0.9262	0.9571	0.8664
Modal	6	0.177	0.0203	0.0002	0	0.9482	0.8693	0	0.0002	0.0001	0.0524	0.9264	0.9572	0.9188
Modal	7	0.17	0.0273	0.0003	0	0.9755	0.8696	0	0.0001	0.0084	0.0271	0.9266	0.9656	0.9458
Modal	8	0.159	0.0068	3.79E-05	0	0.9824	0.8696	0	4.69E-05	0.0187	1.288E-05	0.9266	0.9843	0.9458
Modal	9	0.154	0.0008	0.0088	0	0.9831	0.8784	0	0.0006	2.63E-05	0.005	0.9272	0.9843	0.9508
Modal	10	0.149	0.0005	0.0007	0	0.9837	0.8791	0	2.88E-05	0.0002	0.0018	0.9272	0.9845	0.9526
Modal	11	0.135	0.0053	0.0017	0	0.9889	0.8808	0	1.64E-05	0.002	0.0256	0.9272	0.9866	0.9781
Modal	12	0.139	0	0.1042	0	0.9889	0.985	0	0.0136	4.2E-05	1.299E-06	0.9408	0.9866	0.9781

Centers of Mass and Rigdity

The following Centers of Mass and Rigidity used in the Surakarta City DPRD are shown in Table 14.

Tabel 6. Centers of Mass and Rigdity

TABLE: Centers Of Mass And Rigidity											
Story	**Diaphragm**	**Mass X**	**Mass Y**	**XCM**	**YCM**	**Cum Mass X**	**Cum Mass Y**	**XCCM**	**YCCM**	**XCR**	**YCR**
		kg	kg	m	m	kg	kg	m	m	m	m
Story 2	D1	173253	173253	9.71	11.105	173252.76	173252.76	9.71	11.105		

Period of Vibration to Earthquake Load

The following is the Period of Vibration to the Earthquake Load in the Surakarta City DPRD shown in Table 15.

Table 7. Surakarta City DPRD Office Period

X-Axis Direction

$T_{program} > T$ — $T_{used} = T$

$T_a = < T_{program} = < T$ — $T_{used} = T_{program}$

$T_{a} > T_{program} > T$ — $T_{used} = T_a$

C_u	1,4
T_a	0,303
Tx Program	0.310
$T = C_u.T_a$	0,424
T_{used}	0,310
OK	

Arah Sumbu Y

$T_{program} > T$ — $T_{used} = T$

$T_a = < T_{program} = < T$ — $T_{used} = T_{program}$

$T_{a} > T_{program} > T$ — $T_{used} = T_a$

C_u	1,4
T_a	0,303
Tx Program	0.235
$T = C_u.T_a$	0,424
T_{used}	0,235
OK	

Maximum Deviation Check

The following is a check of the Maximum Deviation based on the ETABS Output at the Surakarta City DPRD Office

Figure 6. Max Displacement Direction X

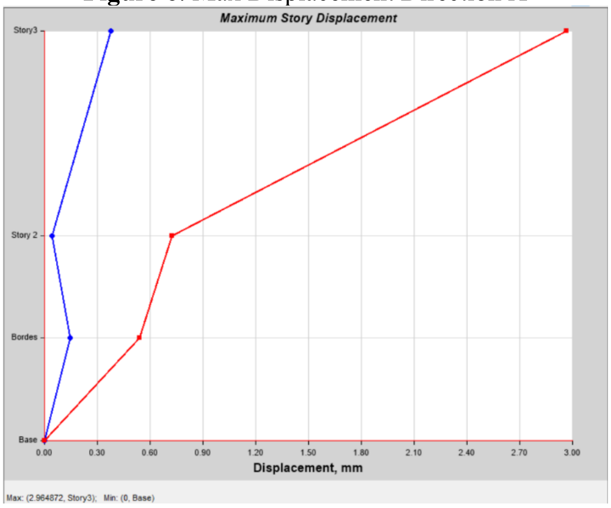


Figure 7. Max Direction Displacement Y

X Directional Deviation Check

The following check of the X deviation at the Surakarta City DPRD Office is shown in Table 8.

Table 8. *X Direction Displacement*

***Story Response Displacement* X**				
Story	***Elevation***	***Location***	**X-Dir**	**Y-Dir**
	m		**mm**	**mm**
Roof	*8*	*Top*	*4.809*	*1.232*
Lt.2	*4*	*Top*	*2.52*	*0.954*
Bordes	*2*	*Top*	*0.536*	*0.115*
LT.1	*0*	*Top*	*0*	*0*

Table 2 Max Directional Spatial X

Story Response EQX							
Story	**hsx**	**h**	**δe**	**Δ**	**Δi**	**Δ ijin**	**Ket**
	(mm)	**(mm)**	**(mm)**	**(mm)**	**(mm)**	**(mm)**	
Roof	8000	4000	4.809	8.8165	6.58E-04	80	**OK**
Lt.2	4000	2000	2.52	4.62	2.85E-05	40	**OK**
Bordes	2000	2000	0.536	0.982666667	6.06E-06	40	**OK**
LT.1	0	0	0	0	0.00E+00	0	**OK**

Table 10. SRPMK Special

KDS D	**Δ ijin /1.3**	**Ket**
Δi		
6.58E-04	26.6666667	**OK**
2.85E-05	13.3333333	**OK**
6.06E-06	13.3333333	**OK**

Y-Directional Deviation Check

The following is a check of the Y deviation at the Surakarta City DPRD Office shown in Table 3.9 to Table 11.

Table 11. Y-Direction Displacement

***Story Response Displacement* Y**				
Story	***Elevation***	***Location***	**X-Dir**	**Y-Dir**
	m		**mm**	**mm**
Roof	*8*	*Top*	*0.376*	*2.965*
Lt.2	*4*	*Top*	*0.043*	*0.723*
Bordes	*2*	*Top*	*0.144*	*0.54*
LT.1	*0*	*Top*	*0*	*0*

Table 13. Max Direction Y Spacing

Story Response EQY							
Story	**hsx**	**h**	**δe**	**Δ**	**Δi**	**Δ ijin**	**Ket**

	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	
Roof	8000	4000	2.965	5.435833333	0.000372	80	**OK**
Lt.2	4000	2000	0.723	1.3255	1.23E-05	40	**OK**
Bordes	2000	2000	0.54	0.99	9.18E-06	40	**OK**
LT.1	0	0	0	0	0	0	**OK**

Table 14. SRPMK Special

KDS D	**Δ ijin /1.3**	**Ket**
Δi		
0.002233	24.7333333	**OK**
4.28E-05	12.3333333	**OK**
9.82E-06	12.3333333	**OK**

X-Direction P-Delta Checking

The following checks of the P-delta working on the structure are shown in Table 3.15

Table 3. P-Delta X Direction

P-DELTA Arah X							
Story	**hsx**	**Δi**	**P**	**Vx**	**Θ x**	**Θ max**	**ket**
	(mm)	**(mm)**	**(kN)**	**(kN)**			
Roof	8000	6.58E-04	37.1234	36.9923	4.50E-08	0.0303	**STABIL**
Lt.2	4000	2.85E-05	123.901	162.7044	2.96E-09	0.0303	**STABIL**
Bordes	2000	6.06E-06	2.6546	64.0563	6.85E-11	0.0303	**STABIL**
LT.1	0	0.00E+00	0	0	0.00E+00	0.03333	**STABIL**

Y-Direction P-Delta Check

The following check of the P-delta acting on the structure is shown in Table 3.13

Table 16. P-Delta Direction Y

P-DELTA Arah Y							
Story	**hsx**	**Δi**	**P**	**Vy**	**Θ y**	**Θ max**	**ket**
	(mm)	**(mm)**	**(kN)**	**(kN)**			
Roof	8000	0.000372	37.1234	40.3799	2.33E-08	0.0303	**STABIL**
Lt.2	4000	1.23E-05	123.901	159.7967	1.30E-09	0.0303	**STABIL**
Bordes	2000	9.18E-06	2.6546	4.26E+01	1.56E-10	0.0303	**STABIL**

LT.1	0	0	0	0	0.00E+00	0.03333	**STABIL**

Conclusion

Kesimpulan	
Syarat Gaya dan Geometri	OK
Kapasitas Lentur	OK
Kapasitas Geser	OK
Kapasitas Torsi	OK
Tulangan Longitudinal	
Longitudinal Tumpuan Atas	2 D18
Longitudinal Tumpuan Tengah	4 D10
Longitudinal Tumpuan Bawah	3 D18
Longitudinal Lapangan Atas	2 D18
Longitudinal Lapangan Tengah	2 D10
Longitudinal Lapangan Bawah	2 D18
Tulangan Transversal/Sengkang	
Sengkang Tumpuan	2D10-99
Sengkang Lapangan	2D10-100

Strong Column Weak Beam (SCWB) Checking

Here's checking Strong Column Weak Beam (SCWB) Floor columns with Beams

Column K1 with B1

Table 17. Checking SCWB K1 with B1 Beam

Pengecekan Strong Column - Weak Beam (SCWB)				
Momen Nominal Kolom, M_{nc}		Input (M_n dari kondisi P_{max} dan P_{min})	kN m	108.851852
M_n^- Tumpuan Balok		Input	kN m	73.128
M_n^+ Tumpuan Balok		Input	kN m	107.401
Cek SCWB	18.7.3.2	$2 * M_{nc} >= 1.2 * (M_n^- + M_n^+)$		OK

Analisis Finite Element Method (FEM)

Inner Style on Column Structure

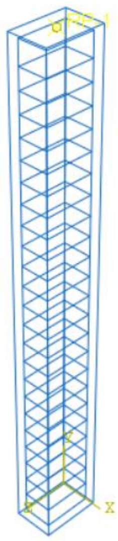
Based on the Inner Style in the Structure of the k1 Column (21 x 38 cm)Surakarta City DPRD Office on the Table 18

Table 18. Inner force on column C52

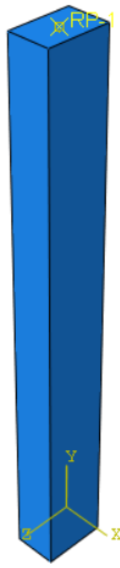
Inner force on column C52		
P	-262.34	kN
M2	-13.255	kN/m
M3	5.4163	kN/m
V2	-2.8397	kN
V3	-8.1367	kN

T	-0.2547	kN/m

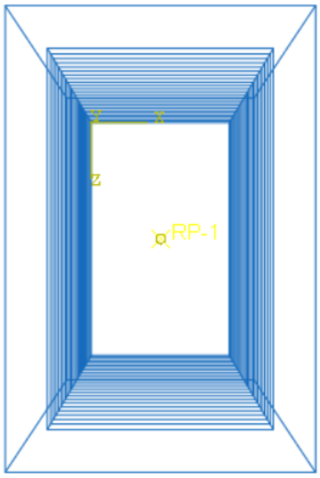
Modeling Existing Columns in Finite Element Method (FEM)



(a). Looks Reinforcement



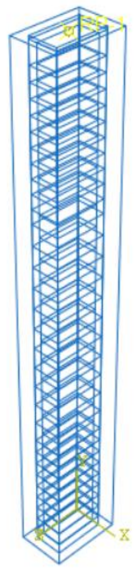
(b) Concrete Modeling



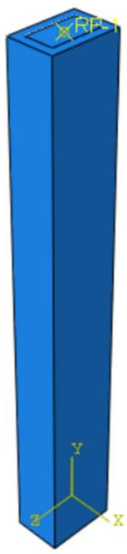
(c) Top View of Rebar and Concrete Columns

Figure 8. Existing Column Modeling

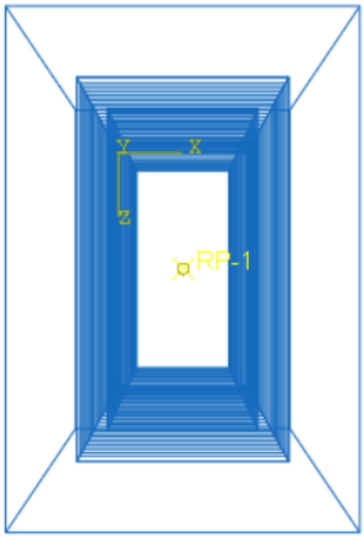
Modeling of 7 cm Jacket Columns on FEM



(a). Looks Reinforcement



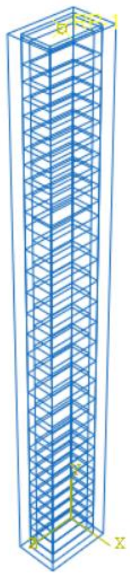
(b) Concrete Modeling



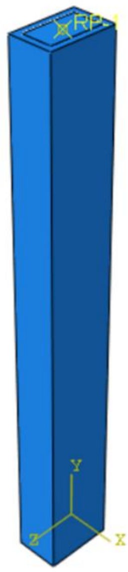
(c) Top View of Rebar and Concrete Columns

Figure 5. 9.Modeling Jacket Column 7 cm

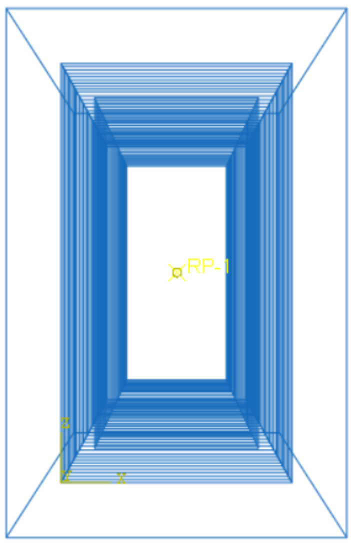
Modeling of 5 cm Jacket Columns on FEM



(a). Looks Reinforcement



(b) Concrete Modeling



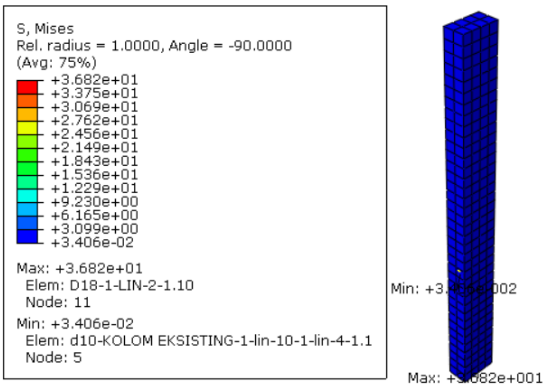
(c) Top View of Rebar and Concrete Columns

Material Properties FEM

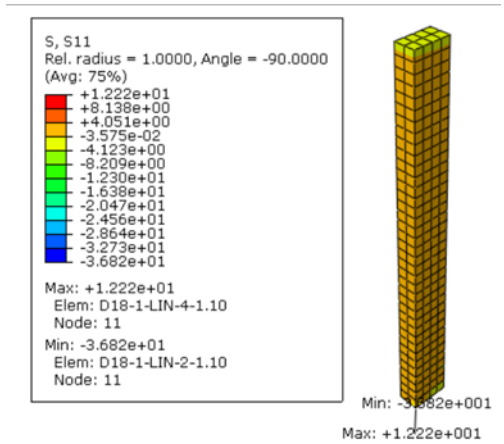
In the modeling of materials in the ABAQUS software, existing concrete with a compressive strength fc = 16 MPa was modeled with a density of 2400 kg/m³, a modulus of elasticity of about 18.8 GPa, and a Poisson ratio of 0.20. For Simulation 2, which is a 7 cm thick concrete jacket with a compressive strength of fc = 35 MPa, an elasticity modulus of around 27.8 GPa is used, while for Simulation 3, a 5 cm thick concrete jacket with fc = 25 MPa has an elasticity modulus of 23.5 GPa. All concrete models use Concrete Damaged Plasticity (CDP) with common parameters in the form of a dilatation angle of 35°, eccentricity of 0.10, a compressive strength to tensile strength ratio of 1.16, a form factor of 2/3, and a viscosity parameter of 0.001 to maintain numerical stability (Qasem et al., 2023).

The reinforcing steel is modeled with a density of 7850 kg/m³, an elasticity modulus of 200 GPa, a Poisson ratio of 0.30, and a melting stress fy = 415 MPa. The relationship between concrete and reinforcement is modeled using embedded constraints so that the behavior of composites can be well represented in simulations (Isleem et al., 2024).

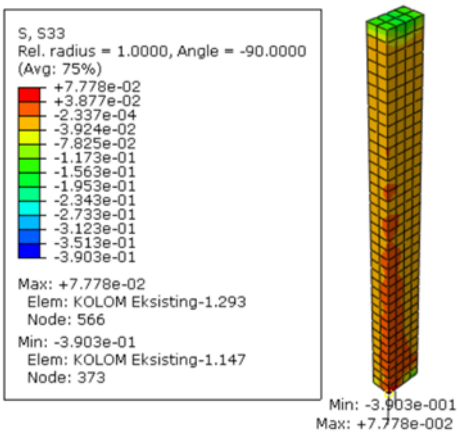
Finite Element Method Analysis Results
Column K1 (21x38 cm) Existing



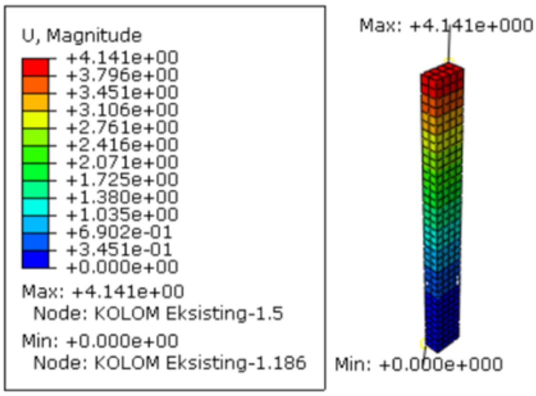
a). Maximum Voltage



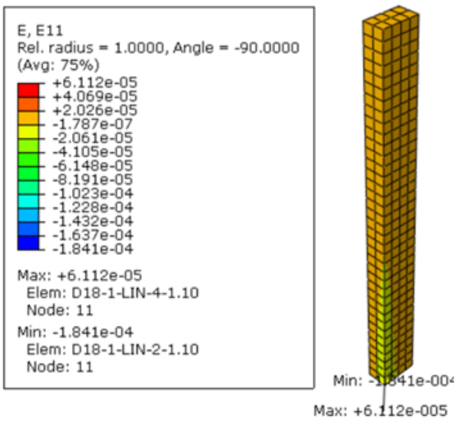
b) Directional Voltage x



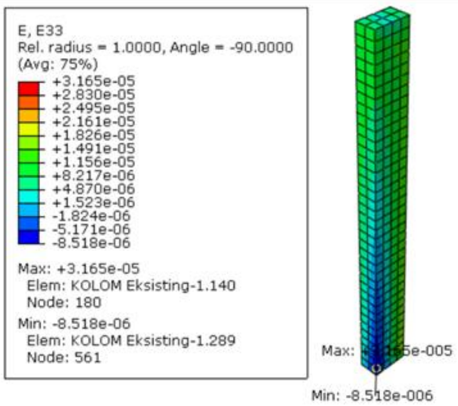
c). Directional voltage y



d). *Displacement* Maksimum



e). Directional Stretch x



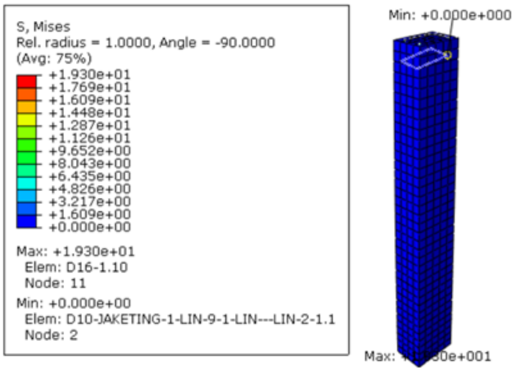
f). Y-Direction Stretch

Figure 10. Analysis Results of Existing Columns

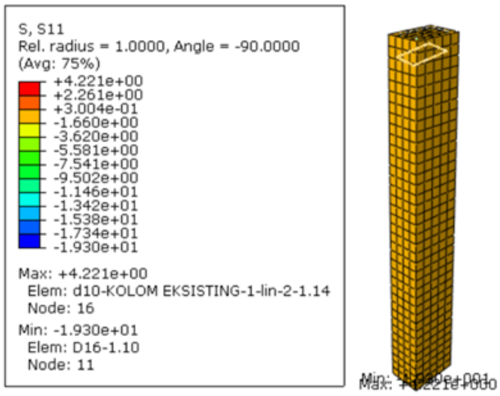
From the results of the analysis, it was obtained that the column received a von Mises voltage of 36.82 MPa, with an X directional voltage of 12.22 MPa and a Y directional voltage of 0.3903 MPa. The value of the X directional strain is 0.00001841, the Y-directional strain is 0.000003165, and the maximum displacement is

4,141 mm. These results show that the column is still working under elastic conditions and that the deformation that occurs is relatively small.

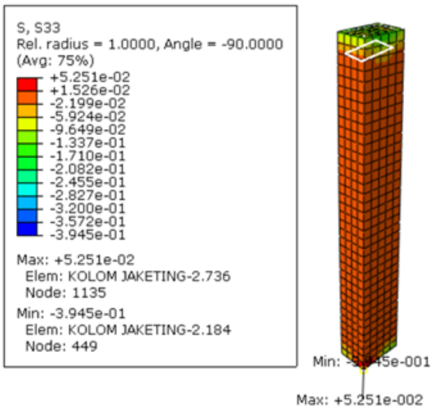
Concrete Coulumn jacketing 7 cm 35 Mpa



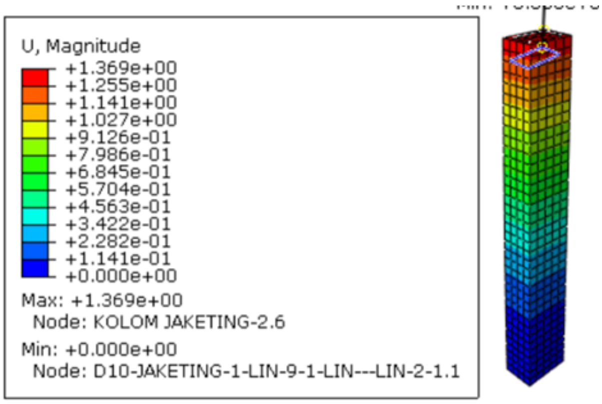
a). Maximum Voltage



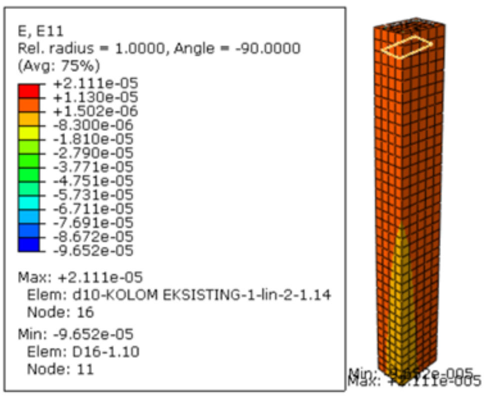
b) Directional Voltage x



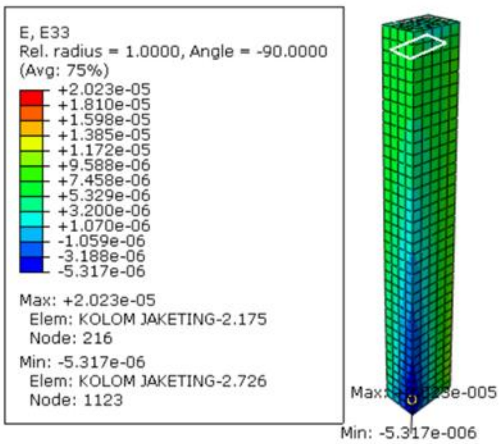
c). Directional voltage y



d). *Displacement* Maksimum



e). Directional Stretch x

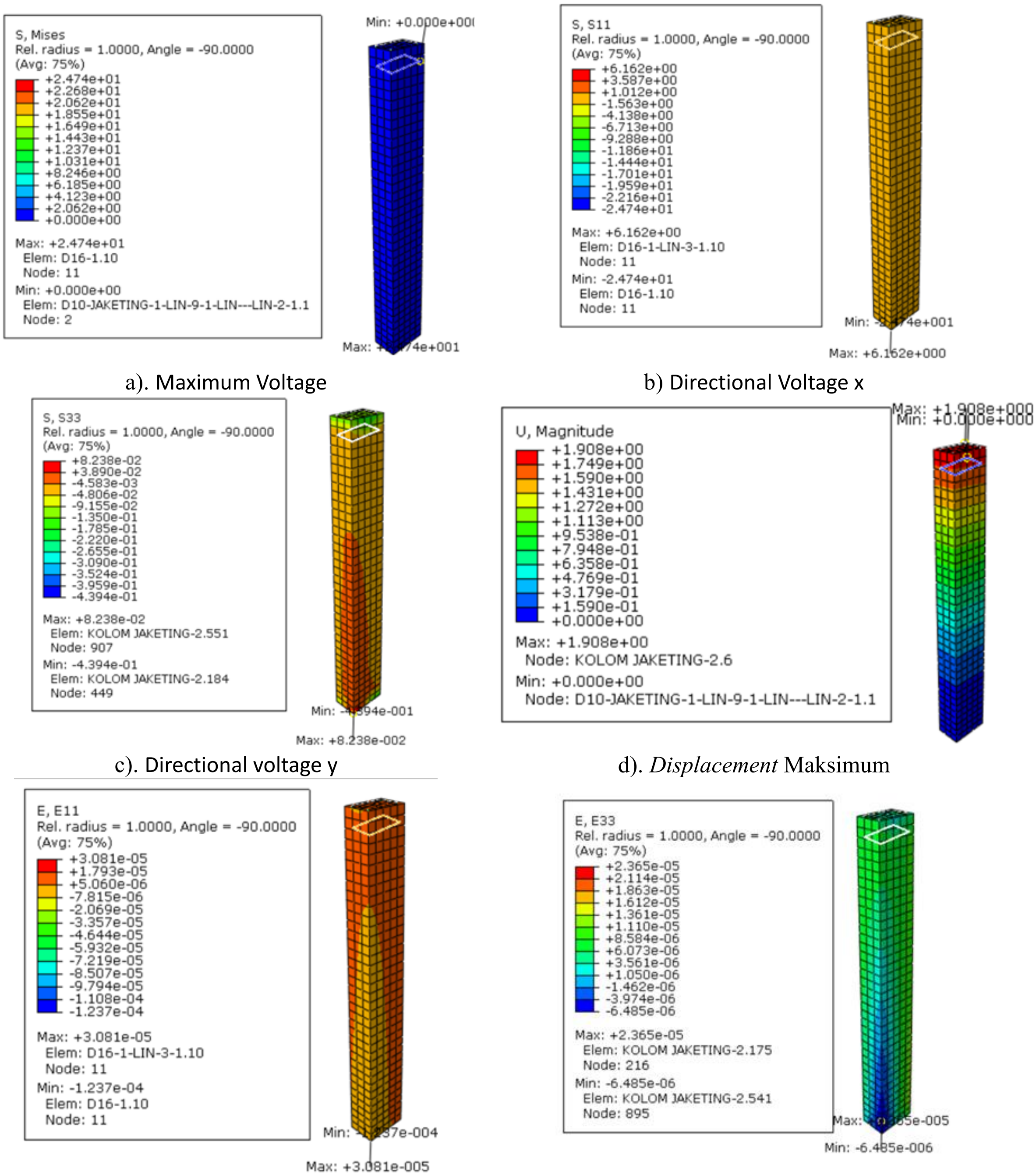


f). Y-Direction Stretch

Figure 11. Jacketing Analysis Results 7 cm

From the results of the analysis, it was obtained that the column received a von Mises voltage of 19.3 MPa, with an X directional voltage of 19.3 MPa and a Y directional voltage of 0.3945 MPa. The value of the X directional strain is 0.000009652, the Y-directional strain is 0.000002023, and the maximum displacement is 1,369 mm. These results show that the column is still in a safe condition with relatively small deformation.

5 cm Jacketing Column with 30 Mpa Concrete



a). Maximum Voltage

b) Directional Voltage x

c). Directional voltage y

d). *Displacement* Maksimum

e). Directional Stretch x

f). Y-Direction Stretch

Figure 12. Analysis Results of Jacketing 5 cm

From the results of the analysis, it was obtained that the column received a von Mises voltage of 24.7 MPa, with an X directional voltage of 6.162 MPa and a Y directional voltage of 0.4394 MPa. The value of the X directional strain is 0.00011237, the Y directional strain is 0.00000265, and the maximum displacement is 1,908 mm. These results show that the column is still in a safe condition with relatively small deformation.

Based on Previous journals

Some studies have shown that concrete jacketing can reinforce existing structural elements, but if the connection between the old concrete and the new jacket (interface) slips or if the surface preparation and bonding agent is inadequate, the stress distribution can become uneven—this has the potential to lead to material fatigue and damaging local deformation. For example, in the study of Suarjana et al. (2020), slip interfaces were found to significantly affect the seismic performance of piers reinforced with concrete jacketing, even though the ultimate capacity increased (Suarjana et al., 2020).

In a critical review by Ishaq & Karim (2024), it was stated that inadequate jacket thickness and poor surface quality can reduce the benefits of jacketing; in extreme conditions, cracks between old concrete and new jackets can appear early, accelerating structural degradation despite the addition of new materials (Ishaq & Karim, 2024).

IV. CONCLUSION

Based on the results of the Hammer Test and Pundit Test, it shows that the quality of concrete in the column structure elements has decreased strength and inconsistency. The results of visual observations also show that most of the columns have experienced significant cracks, especially in the area of connections and confluences between structural elements.

The results of structural modeling analysis using ETABS show that in the second shape mode there is a global rotation that should indicate translation, indicating a weakness in the lateral rigidity system of the structure. In addition, the column capacity ratio showed overstress conditions in 26 out of a total of 38 columns, which means that more than 68% of column elements did not meet the design capacity. In terms of dimensions, the short side of the existing column is only 210 mm, while according to SNI 2847:2019 Article 18.7.2.1, the minimum dimension of the column for multi-storey buildings must be ≥ 300 mm. This condition indicates that the existing column elements do not meet the minimum requirements for compressive capacity or lateral stability.

From the results of the Finite Element Method (FEM) analysis, the existing conditions show that the column receives a von Mises voltage of 36.82 MPa, with an X directional voltage of 12.22 MPa and a Y direction of 0.3903 MPa, and a maximum displacement of 4,141 mm. This indicates that although the columns are still in an elastic condition, the structure has shown indications of working close to the limit of material capacity.

Reinforcement simulation with the concrete jacketing method showed an increase in the capacity of the structure. In 7 cm jacketing using fc 35 MPa quality concrete with D16-4 reinforcement and Ø10-100 mm stubble, the von Mises stress decreased to 19.3 MPa and the deformation decreased to 1,369 mm. Meanwhile, in 5 cm jacketing with 30 MPa fc concrete, the von Mises voltage was recorded at 24.7 MPa with a deformation of 1,908 mm. These results show that theoretically, reinforcement with jacketing is able to significantly increase the capacity of the structure.

However, the existing conditions show that many cracks have spread to the old concrete, which can technically degrade the quality of the bond between the old concrete and the new concrete. This has the potential to cause slip in the contact field and decrease the effectiveness of reinforcement. Based on these conditions, the jacketing method is considered to have high technical risks and does not provide a guarantee of optimal strength increase.

Taking into account the results of material testing, widespread cracking conditions, column dimensions that do not meet standards, and the results of numerical analysis that show many column elements in overstress conditions, the most rational and safe alternative is to demolish and rebuild the structure. This approach will result in a new structure system that meets the design requirements according to SNI 2847:2019 and SNI 1726:2019, as well as ensuring the safety and performance of the structure in the long term.

REFERENCES

[1]. Faisal Ridho, H. K. (2015). PERBANDINGAN MUTU BETON HASIL UPVT METODE INDIRECT TERHADAP MUTU BETON HASIL HAMMER TEST DAN CORE DRILL. Jurnal Konstruksia, Volume 6, 25–39.

[2]. Fauzan, Hakam, A., Ismail, F. A., Jauhari, Z. Al, & Agista, G. A. (2024). Seismic Damage Assessment and Retrofitting of a Hospital Building in West Pasaman, Indonesia. International Journal of GEOMATE, 27(120), 122–129. https://doi.org/10.21660/2024.120.g13339

[3]. Goro, G. L., Irsyam, M., Meilano, I., & Asrurifak, M. (2024). Comparative Study of Seismic Hazard Analysis Using Classical PSHA and PSHA Method in Java Island, Indonesia. Indonesian Journal on Geoscience, 11(3), 349–363. https://doi.org/10.17014/ijog.11.3.349-363

[4]. Herdiansah, Y., & Cendana, W. P. (2025). Evaluasi Kuat Tekan Beton Terpasang pada Struktur Dermaga Eksisting Berdasarkan Pengujian Destructive dan Non-Destructive. Jurnal Teknik Sipil, 21(1), 109–127. https://doi.org/10.28932/jts.v21i1.8698

[5]. Hidayat, N., & Santoso, E. W. (1997). Gempa Bumi Dan Mekanismenya. In Alami: Jurnal Teknologi Reduksi Resiko Bencana (Vol. 2, Issue 3, p. 50).

[6]. Holmes, J. D.; Weller, R.; and Richards, P. (2002). Design Wind Speeds for the Asia-Pacific Region. Standards Australia.
[7]. Husein, S. (2016). Bencana Gempabumi. DRR Action Plan Workshop: Strengthened Indonesian Resilience: Reducing Risk from Disasters, January, 1–10. https://doi.org/10.13140/RG.2.1.1112.6808
[8]. Ishaq, M. B., & Karim, F. R. (2024). Effect of Thickness of Reinforced Concrete Jacketing on Solid Reinforced Concrete Beam’s Flexural Strength - A Critical Review. Construction, 4(2), 94–102. https://doi.org/10.15282/construction.v4i2.10687
[9]. Isleem, H. F., Chukka, N. D. K. R., Bahrami, A., Kumar, R., & Sor, N. H. (2024). Nonlinear finite element and machine learning modeling of tubed reinforced concrete columns under eccentric axial compression loading. Alexandria Engineering Journal, 92(December 2023), 380–416. https://doi.org/10.1016/j.aej.2024.02.026
[10]. Mahmoudi, P., Asadi, P., & Eftekhar, M. R. (2026). Seismic Vulnerability Assessment and Retrofit Strategies for Masonry Vaults. 39(07), 1778–1789.
[11]. Nastri, E. (2025). Performance-Based Seismic Design. 4–9.
[12]. Pusat Studi Gempa Nasional (PuSGeN). (2017). PERAN PUSGEN DALAM TATA KELOLA KESELAMATAN DAN KEAMANAN INFRASTRUKTUR BANGUNAN TAHAN GEMPA. Pusat Studi Gempa Nasional (PuSGeN), Kementerian PUPR (Pekerjaan Umum dan Perumahan Rakyat).
[13]. Qasem, M., Hasan, M., & Muhamad, R. (2023). Finite element simulation of precast moment resisting reinforced concrete beam-column joint subjected to monotonic load. Materials Today: Proceedings, xxxx. https://doi.org/10.1016/j.matpr.2023.03.532
[14]. Rohman, R. K. (2025). Seismic Evaluation of the Magetan DPRD Building in East Java , Indonesia Using Pushover Analysis. 15(5), 26423–26429.
[15]. Soehaimi, A. (2008). Seismotektonik dan Potensi Kegempaan Wilayah Jawa. Indonesian Journal on Geoscience, 3(4), 227–240. https://doi.org/10.17014/ijog.3.4.227-240
[16]. Suarjana, M., Octora, D. D., & Riyansyah, M. (2020). Seismic performance of rc hollow rectangular bridge piers retrofitted by concrete jacketing considering the initial load and interface slip. Journal of Engineering and Technological Sciences, 52(3), 343–369. https://doi.org/10.5614/j.eng.technol.sci.2020.52.3.4
[17]. Wongso Suharto, S., & Fuk Jin, O. (2024). Penerapan Metode Pembongkaran (Demolishing) Pada Bangunan Gedung dan Daur Ulang Limbah Bongkaran: A Systematic Literature Review. Teras Jurnal : Jurnal Teknik Sipil, 14(1), 223–236. https://doi.org/10.29103/tj.v14i1.1059
[18]. Zachari, M. Y., & Turuallo, G. (2020). Analisis Struktur Baja Tahan Gempa dengan Sistem SRPMK (Struktur Rangka Pemikul Momen Khusus) Berdasarkan SNI 1729:2015 dan SNI 1726:2012. REKONSTRUKSI TADULAKO: Civil Engineering Journal on Research and Development, September, 9–16. https://doi.org/10.22487/renstra.v1i2.24
[19]. Zhang, L., Hou, Y., & Wang, Y. (2025). Finite Element Analysis of the Mechanical Performance of an Innovative Beam-Column Joint Incorporating V-Shaped Steel as a Replaceable Energy-Dissipating Component. Buildings, 15(14), 1–25. https://doi.org/10.3390/buildings15142513