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Pushover Analysis Based to ASCE 41-17 of Islamic Center Tower on North Morowali Using Shear Wall

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Abstract: Earthquake on 28th September 2018 with magnitude 7,4 Mw caused massive destructions especially buildings had collapsed. The North Morowali located between active Weluki and Matano Faults is also not safe from earthquake disaster hazard. Therefore, the tower structure of the Islamic Center, using shear wall system, needs to be designed based on SNI 1726-2019 and its performance evaluated using pushover analysis based to ASCE 41-17. The target displacement for the shear wall system (δ_T) in the X direction is 0.04465557 m and in the Y direction is 0.0446518 m. The performance level based on ASCE 41-17 for the tower structure with a shear wall system is LS (Life Safety) in both the X and Y directions.

Keywords: *Earthquake, Pushover Analysis, ASCE 41-17, Shear Wall, Target Displacement, Performance Level*

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I. INTRODUCTION

North Morowali Regency is one of the largest nickel-producing regencies in Central Sulawesi. As a result, North Morowali Regency has a high rate of economic growth. In addition, Kolonodale—the capital of North Morowali Regency—is geographically situated near the sea and flanked by mountains, leaving very little flat land. Furthermore, North Morowali Regency is flanked by two active faults: the Matano Fault (sliding at 30–44 mm/year) and the Weluki Thrust Fault (Figure 1). Due to these geographical conditions, many future buildings—particularly government buildings and public service facilities—will be constructed vertically.

As a result of this accelerating economic growth, an increasing number of people from outside the region are moving to North Morowali Regency. To address this challenge, the local government, as a public servant, is taking steps to improve the quality of public services. As part of its commitment to improving the quality of public services, the local government will construct an Islamic Center in Kolonodale, the capital of North Morowali Regency. The main building of the Islamic Center will be 12.30 meters tall, while its minaret will stand 27.30 meters high. This facility is expected to serve as a safe, durable, and comfortable hub for religious activities for the community

Following the natural disaster that occurred on September 28, 2018, in which the city of Palu, Sigi Regency, and Donggala were struck by a 7.4 Mw earthquake, many buildings were severely damaged, resulting in numerous casualties. The earthquake was a tectonic event caused by plate movement along the Palu-Koro fault, releasing 7.4 Mw of energy at a depth of 10 km—classified as a shallow earthquake. Given that North Morowali Regency is situated between two faults, there is a need for buildings that offer structural stability, durability, and comfort for their occupants.

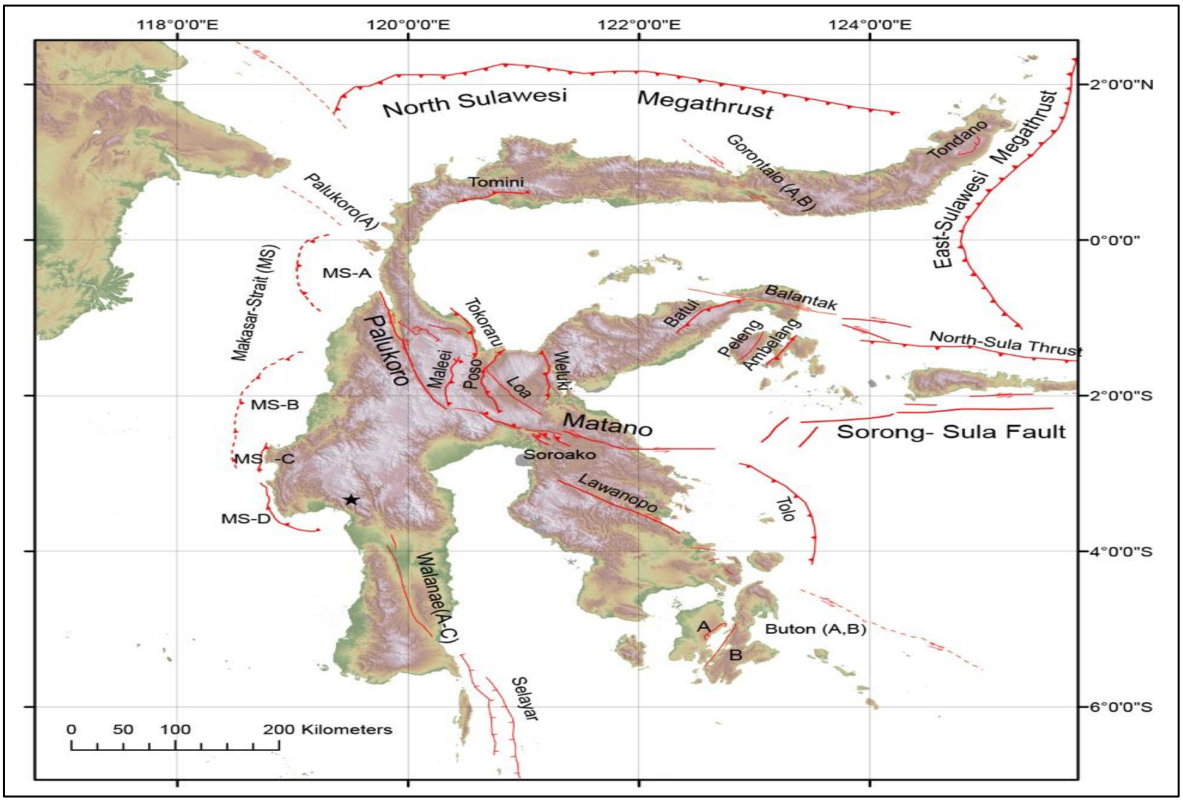

Fig 1 Active Fault Map on Sulawesi (*)

A. Performance-Based Seismic Design

Performance-based seismic design is a design concept aimed at predicting the performance of building structures during an earthquake. When it was first introduced, the Performance-Based Seismic Design approach was intended to evaluate existing buildings for rehabilitation and retrofitting—that is, to strengthen building structures before or after an earthquake so that they can withstand a specific earthquake intensity.

In performance-based design, the performance objectives of a building with respect to earthquakes are clearly stated so that owners, tenants, insurers, government agencies, or funders have the opportunity to specify the desired conditions; these specifications are then used by designers as guidelines. Performance objectives consist of a specified design earthquake hazard and the allowable damage level, or performance level, of the building in response to that earthquake. Referring to FEMA-273 (1997), which serves as the classic reference for performance-based design, the categories of structural performance levels are [2] :

- Immediate Occupancy (IO),
- Life-Safety (LS),
- Collapse Prevention (CP).

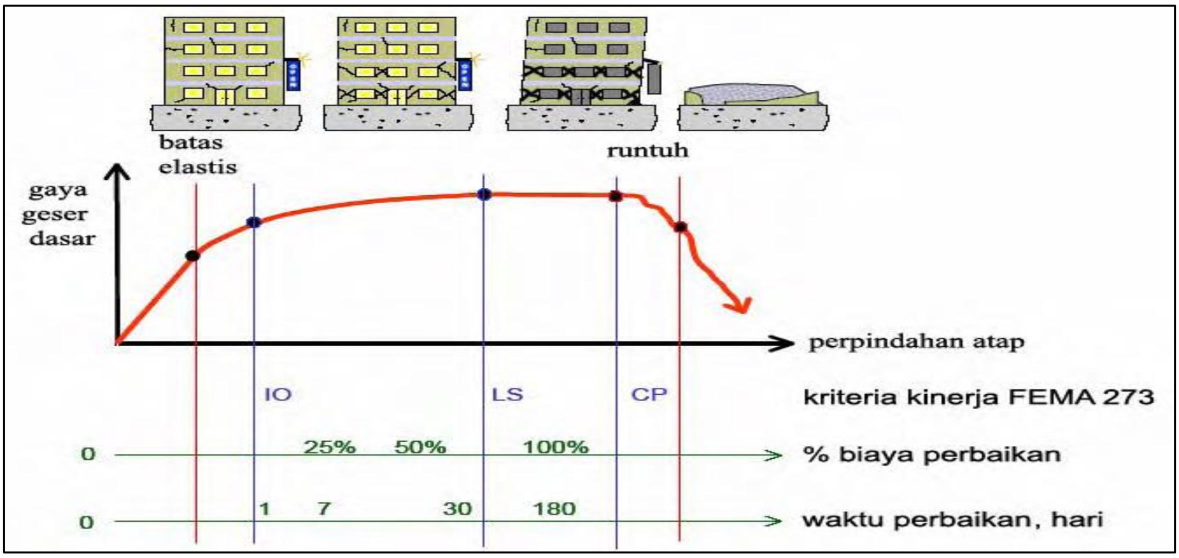

Fig 2 Illustration of Performance-Based Seismic Engineering [1]

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B. Pushover Analysis

Pushover analysis in ASCE 41-17 uses the displacement coefficient method, which is intended to represent the maximum displacement that occurs at a specific seismic hazard level [3]. In the pushover analysis, the structure is subjected to a load until it yields at one or more locations within the structure. The capacity curve will exhibit linear behavior before reaching the yield point and will subsequently exhibit nonlinear behavior..

The purpose of pushover analysis is to estimate the maximum forces and deformations that occur and to identify which components are critical. This allows for the identification of components that require special attention in terms of detailing or stability. Numerous studies have shown that pushover analysis can provide adequate results (when compared to the results of nonlinear dynamic analysis) for regular, low-rise buildings [2].

As the load on the structure is gradually increased, the structural members will undergo plasticization, resulting in the formation of plastic hinges. A plastic hinge is a process that occurs at the ends of structural members, where they transition from rigid behavior to that of a hinge as the structure dissipates seismic energy through its members. To evaluate a structure, plastic hinge modeling is performed on structural members expected to undergo plasticization. The objective is to describe the nonlinear behavior of the structural elements. Plastic hinges can be modeled using auto-hinges, where the backbone curve is created based on ASCE 41-17 Table 10-7 for columns with DOFs P-M2-M3 and beams with DOFs M2 and M3[4].

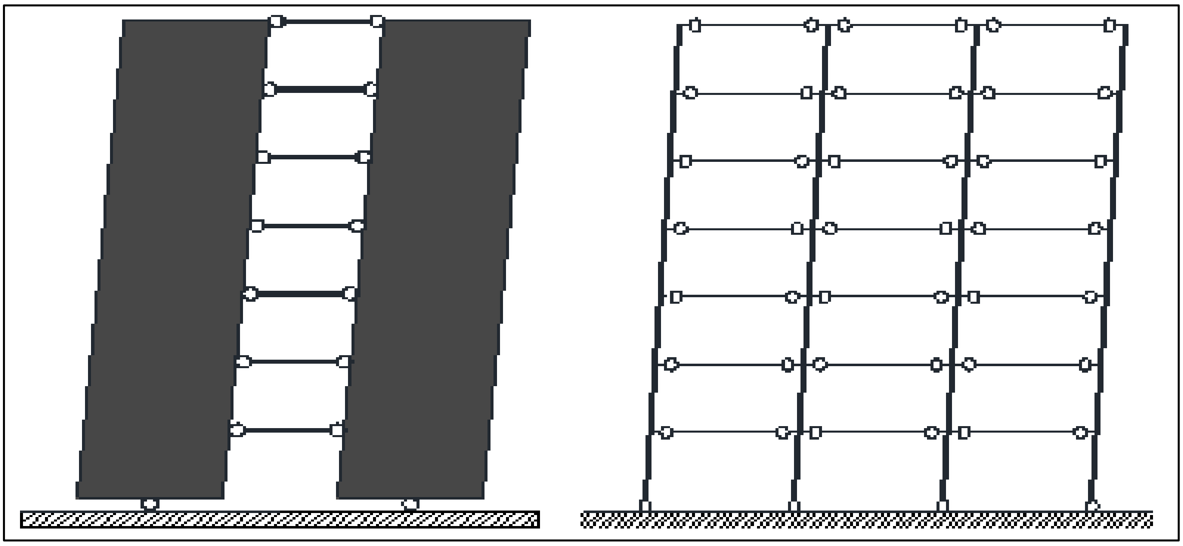

Fig 3 Ideal Plasticization Mechanism in Structures [5]

C. Displacement Target

The forces and deformations of each component/element are calculated relative to a specific displacement at a control point, referred to as the target displacement (denoted by δ_t), which is considered the maximum displacement that occurs when the building experiences the design earthquake. To determine the structure's post-collapse behavior, a pushover analysis must be performed to generate a curve showing the relationship between the base shear force and the lateral displacement at the control point up to at least 150% of the target displacement [2]. According to ASCE 41-17, the displacement target (δ_t) can be calculated using the formula [6]:

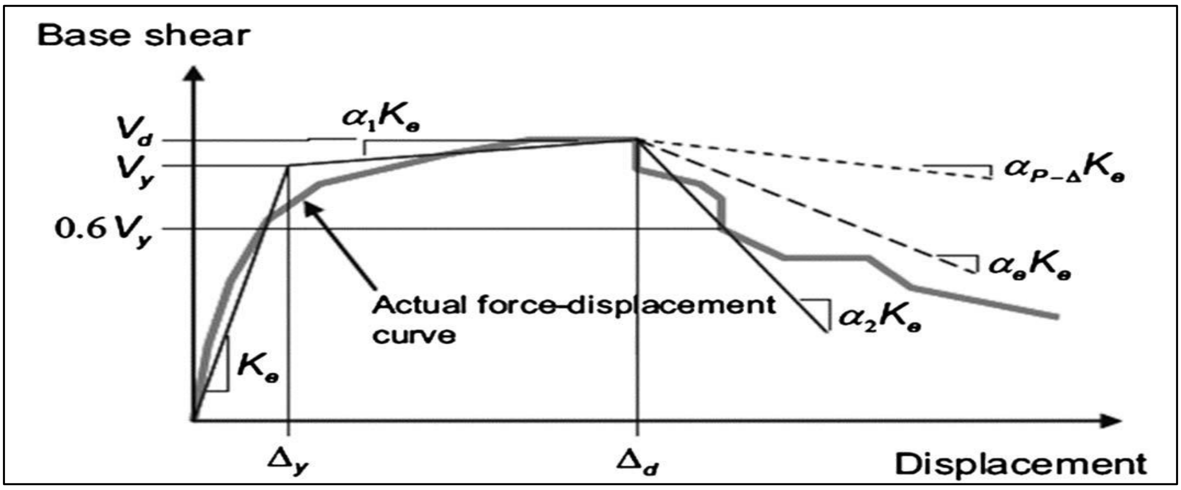

Fig 4 Base Shear – Displacement Relationship [7]

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$$\delta_T = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g \quad (1)$$

➢ *Notations:*

- δ_T : Displacement target (m)
- C_0 : Modification factor for spectral displacement relative to roof displacement
- C_1 : Modification factor for maximum inelastic displacement relative to elastic displacement
- C_2 : Modification factor represents the pinched hysteresis effect
- S_a : Acceleration spectrum response at the effective fundamental period
- T_e : Effective fundamental period (s)
- g : Gravity acceleration (m/s^2)

II. METHODOLOGY

A. Location

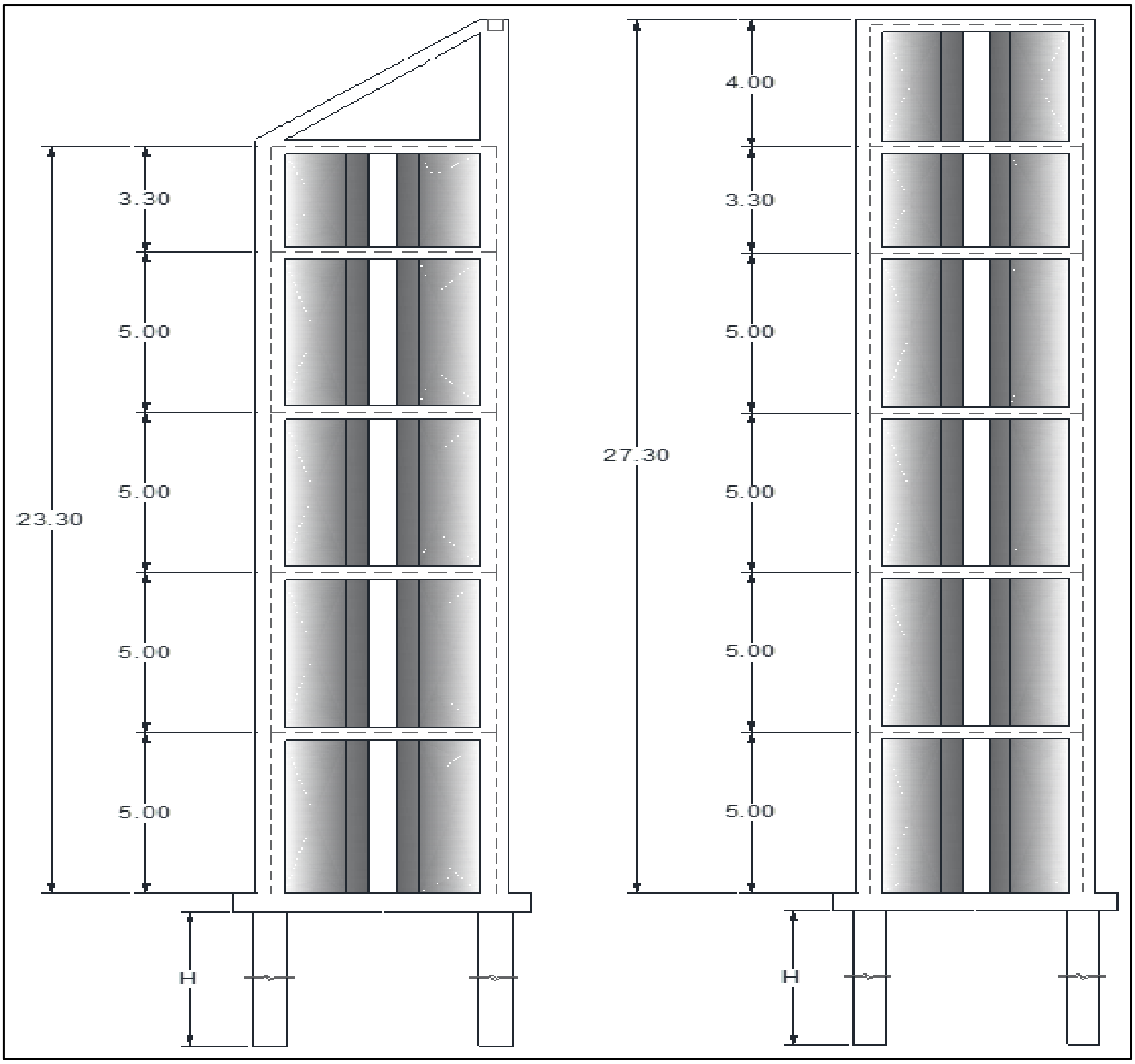
The Islamic Center tower, which is the subject of this study, is located on Pelita Street in Bahontula Village, Petasia Subdistrict, North Morowali Regency, Central Sulawesi, at coordinates 1°59'0.71"S latitude and 121°20'29.39"E longitude. The Islamic Center building falls under the jurisdiction of the North Morowali Regency Government. This study is scheduled for September 2024 and will examine the design plans developed by the Islamic Center's architects.

B. Preliminary Design

For the preliminary structural design, data provided by the structural engineer specifically the dimensions of columns, beams, and slabs is used. Meanwhile, the preliminary design of shear walls and portal bracing is carried out in accordance with the provisions of SNI 1726:2019 and SNI 2847:2019.

➢ *In Addition, the Properties of the Concrete and Steel Used in this Analysis are as Follows:*

- Quality of concrete (f'$_c$) : 25 MPa
- Quality of Main Reinforcing Steel (f_{yu}) : 420 MPa
- Quality of Hoop Reinforcing Steel (f_{yp}): 280 MPa



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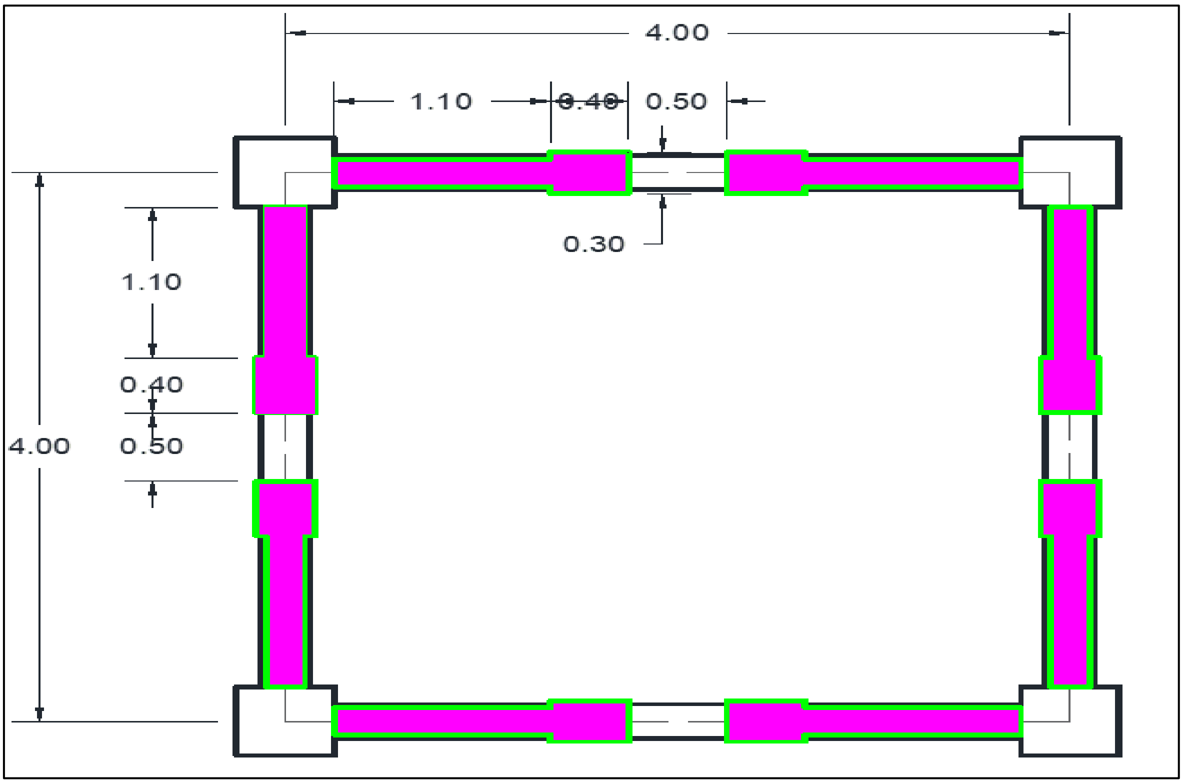

Fig 5 Tower Structure Model

Table 1 Structure Dimension Elements

No	Elements	Dimension (m)
1	Column 1st Floor - Roof	0,50 x 0,50
2	Floor Beam 2nd – 5th	0,30 x 0,45
3	Roof Beam	0,25 x 0,40
4	*Boundary Element*	0,30 x 0,40
5	*Wall Thickness*	0,20
6	Floor Plate	0,14
7	Roof Plate	0,12

C. Structural Modeling

To analyze and design the tower structure, ETABS version 22.0.0 was used as a tool. The software was used to model the existing structure in accordance with SNI 1726:2019 and SNI 2847:2019.

D. Structural Load

When calculating the self-weight of a structure, the ETABS software automatically calculates the weights of columns, beams, and slabs. The loads that need to be entered manually are the superimposed dead load and the live load.

➢ *Floor Plate Load*

- *Super Imposed Dead Load*

✓ Brick wall plastered on both sides = 2.50 kN/m^2
✓ Ceramics = 1.10 kN/m^2
✓ Mechanical and Electrical = 0.25 kN/m^2
✓ Plafond = 0.78 kN/m^2

- *Live Load*

✓ Religion place = 6.00 kN/m^2

➢ *Roof Plate Load*

- *Super Imposed Dead Load*

✓ Mechanical and Electrical = 0.25 kN/m^2
✓ Plafond = 0.78 kN/m^2
✓ Waterproofing bitumen = 1.50 kN/m^2

- *Live Load*

✓ Religion place = 0.96 kN/m^2
✓ Rain 5 cm = 1.46 kN/m^2

E. Seismic Loads Using the Response Spectrum Method

To find out the design spectrum response for a building site, visit the website https://rsa.ciptakarya.pu.go.id/2021/ online. After entering the tower's location coordinates and setting the soil type to Soft Soil (SE), the design spectrum response data is obtained, as shown in Figure 6.

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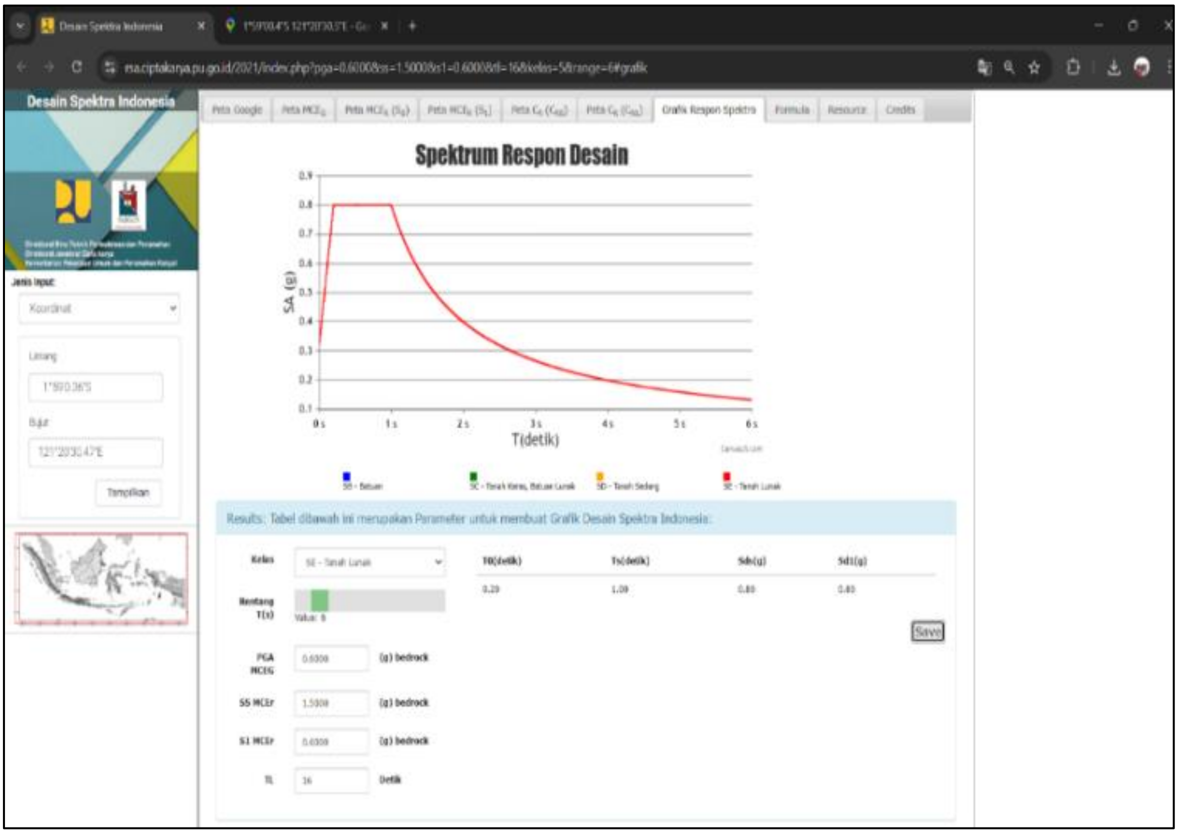

Fig 6 Design Response Spectrum Based on Coordinates

From the Figure 6, the spectral response parameters for acceleration can be calculated as follows:

PGA	=	0,60 g
S_S	=	1,50 g
S_1	=	0,60 g
T_0	=	0,20 s
T_s	=	1,00 s
T_L	=	16,00 s
S_{DS}	=	0,80 g
S_{D1}	=	0,80 g
F_a	=	0,80
F_v	=	2,00
S_{MS}	=	$F_a \, x \, S_S = 0,80 \, x \, 1,50 = 1,20$
S_{M1}	=	$F_v \, x \, S_1 = 0,60 \, x \, 2,00 = 1,20$

F. Load Combinations

Referring to SNI 1727-2020 regarding the minimum design loads for buildings, the ultimate load combination used is as follows:

a) Comb.1 : 1,4 DL
b) Comb.2 : 1,2 DL + 1,6 LL
c) Comb.3 : 1,36 DL + 1,0 LL + 1,3 Ex + 0,39 Ey
d) Comb.4 : 1,36 DL + 1,0 LL + 1,3 Ex – 0,39 Ey
e) Comb.5 : 1,36 DL + 1,0 LL – 1,3 Ex + 0,39 Ey
f) Comb.6 : 1,36 DL + 1,0 LL – 1,3 Ex – 0,39 Ey
g) Comb.7 : 1,36 DL + 1,0 LL + 0,39 Ex + 1,3 Ey
h) Comb.8 : 1,36 DL + 1,0 LL + 0,39 Ex – 1,3 Ey
i) Comb.9 : 1,36 DL + 1,0 LL + 0,39 Ex – 1,3 Ey
j) Comb.10 : 1,36 DL + 1,0 LL – 0,39 Ex + 1,3 Ey
k) Comb.11 : 0,74 DL + 1,3 Ex + 0,39 Ey
l) Comb.12 : 0,74 DL + 1,3 Ex – 0,39 Ey
m) Comb.13 : 0,74 DL – 1,3 Ex + 0,39 Ey
n) Comb.14 : 0,74 DL – 1,3 Ex – 0,39 Ey
o) Comb.15 : 0,74 DL + 0,39 Ex + 1,3 Ey
p) Comb.16 : 0,74 DL + 0,39 Ex – 1,3 Ey
q) Comb.17 : 0,74 DL – 0,39 Ex + 1,3 Ey
r) Comb.18 : 0,74 DL – 0,39 Ex – 1,3 Ey

G. Modal Participating Mass

In accordance with the requirements of SNI 1726-2019, Section 7.9.1.1 stipulates that an analysis must be conducted to determine the natural vibration modes of the structure. The analysis must achieve a participating mass ratio of at least 90% of the actual mass in each direction under consideration. The analysis results show that the participating mass ratio exceeds 90%, thereby meeting the requirements specified in SNI 1726-2019.

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Table 2 Modal Participating Mass Ratio

Case	Mode	Period sec	UX	UY	UZ	SumUX	SumUY	SumUZ	RX	RY	RZ	SumRX	SumRY	SumRZ
Modal	1	0.443	0.000	0.745	0.000	0.000	0.745	0.000	0.399	0.000	0.004	0.399	0.000	0.004
Modal	2	0.442	0.748	0.000	0.000	0.748	0.745	0.000	0.000	0.397	0.000	0.399	0.397	0.004
Modal	3	0.243	0.000	0.004	0.000	0.748	0.749	0.000	0.001	0.000	0.823	0.400	0.397	0.827
Modal	4	0.115	0.000	0.170	0.000	0.748	0.919	0.000	0.410	0.000	0.002	0.810	0.397	0.828
Modal	5	0.114	0.173	0.000	0.000	0.922	0.919	0.000	0.000	0.419	0.000	0.811	0.816	0.828
Modal	6	0.074	0.000	0.003	0.000	0.922	0.922	0.000	0.006	0.000	0.105	0.817	0.816	0.934
Modal	7	0.055	0.039	0.000	0.000	0.961	0.922	0.000	0.000	0.079	0.000	0.817	0.895	0.934
Modal	8	0.055	0.000	0.042	0.000	0.961	0.964	0.000	0.087	0.000	0.003	0.903	0.895	0.937
Modal	9	0.045	0.015	0.000	0.000	0.976	0.964	0.000	0.000	0.037	0.000	0.903	0.932	0.937
Modal	10	0.041	0.000	0.011	0.000	0.976	0.975	0.000	0.028	0.000	0.027	0.932	0.932	0.963
Modal	11	0.037	0.000	0.009	0.000	0.976	0.984	0.000	0.026	0.000	0.016	0.958	0.932	0.979
Modal	12	0.032	0.019	0.000	0.000	0.994	0.984	0.000	0.000	0.055	0.000	0.958	0.986	0.979
Modal	13	0.029	0.000	0.013	0.000	0.994	0.997	0.000	0.034	0.000	0.000	0.992	0.986	0.979
Modal	14	0.027	0.000	0.002	0.000	0.994	0.999	0.000	0.005	0.000	0.010	0.997	0.986	0.989
Modal	15	0.026	0.006	0.000	0.000	1.000	0.999	0.000	0.000	0.013	0.000	0.997	1.000	0.989
Modal	16	0.024	0.000	0.001	0.000	1.000	1.000	0.000	0.002	0.000	0.008	0.999	1.000	0.997

H. Nonlinear Static Analysis (Pushover)

Once a safe tower structure has been designed in accordance with the standards set forth in SNI, the structure is then simulated to be subjected to gradual and monotonic seismic forces in a single direction until it experiences fatigue and collapse in certain parts of the structure.

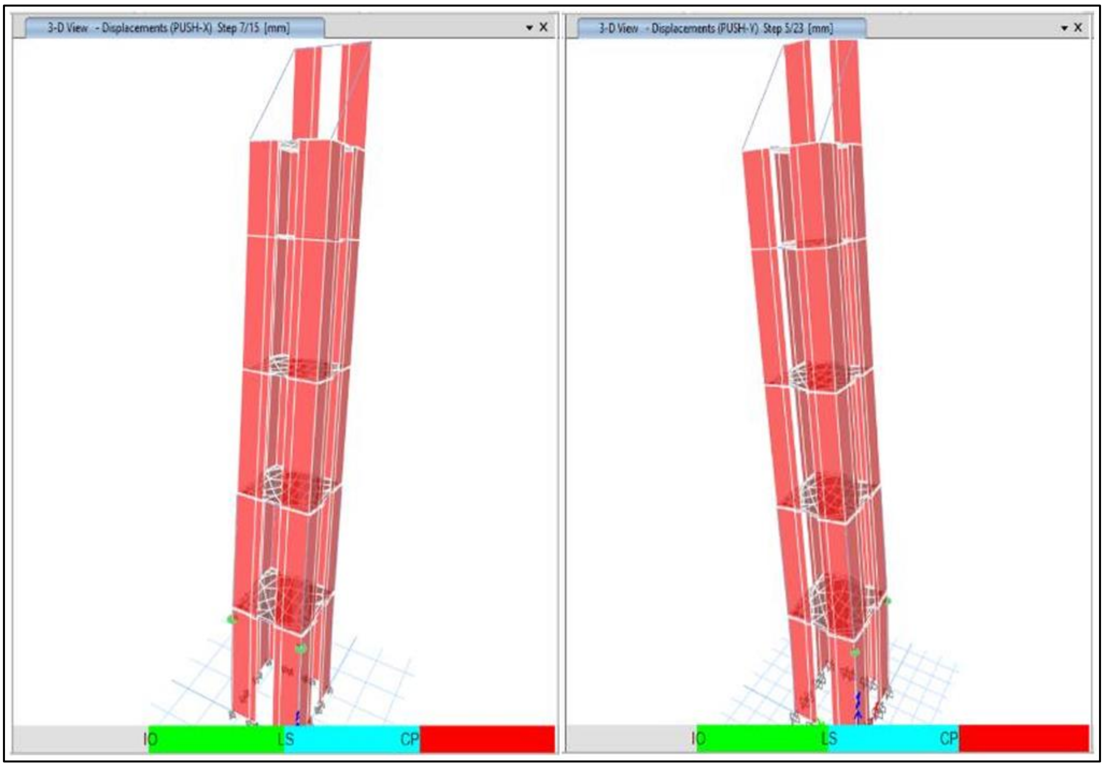

Fig 7 Nonlinear Static Analysis (Pushover)

III. RESULT AND DISCUSSION

A. Inter-Story Deflections

In accordance with the requirements of SNI 1726-2019, Section 7.12.1, regarding the limits on inter-story deflection, it is specified that the deflection occurring between stories of a structure must not exceed the allowable inter-story deflection (Δ_a) as set forth in Table 2.15. In compliance with the building's risk category such as Category IV, the allowable inter-story deflection is $\Delta_a = 0.010 \times hsx$. The inter-story deflections for building structures with a shear wall system are shown in the following table 3.

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Table 3 Inter Story Deflection X Direction

Story	Height (m)	Deflection (m)	Allowable Deflection, Δ_a (m)	Description
5	23.3	0.0147	0.0330	Safe
4	20.0	0.0126	0.0500	Safe
3	15.0	0.0090	0.0500	Safe
2	10.0	0.0051	0.0500	Safe
1	5.0	0.0017	0.0500	Safe

Table 4 Inter Story Deflection Y Direction

Story	Height (m)	Deflection (m)	Allowable Deflection , Δ_a (m)	Description
5	23.3	0.0148	0.0330	Safe
4	20.0	0.0126	0.0500	Safe
3	15.0	0.0090	0.0500	Safe
2	10.0	0.0051	0.0500	Safe
1	5.0	0.0017	0.0500	Safe

B. Target Displacement

➢ *X Direction*

The results from the ETABS analysis provided a pushover capacity curve as shown in Figure 8.

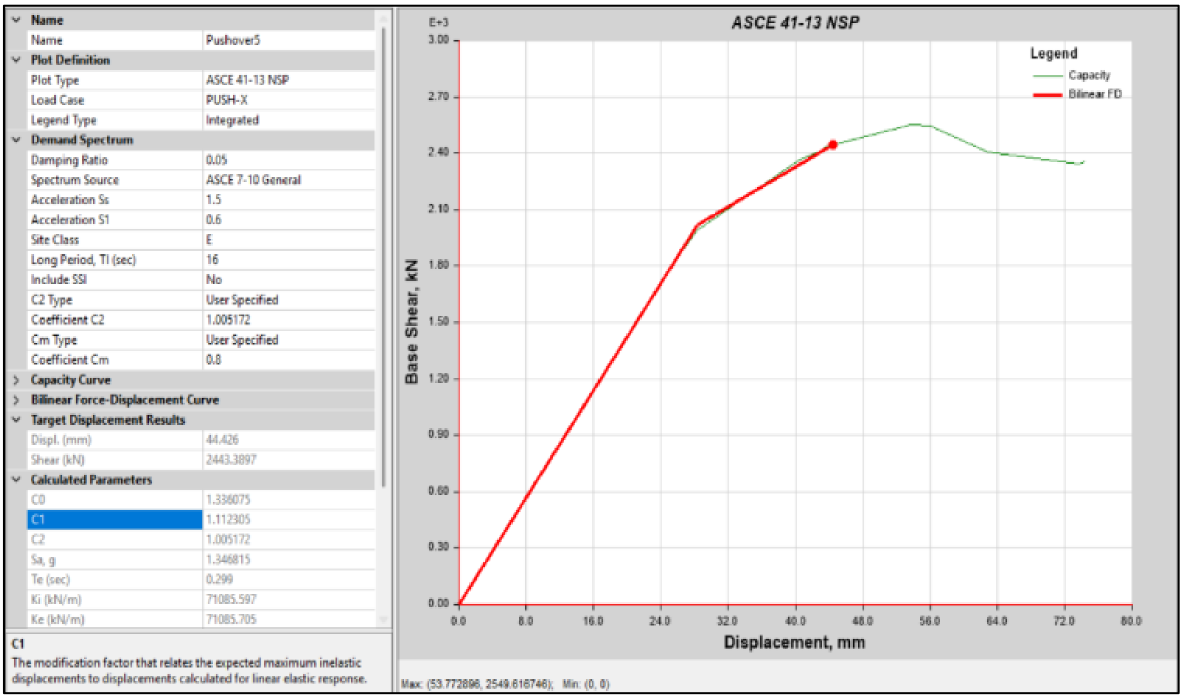

Fig 8 Curve Between Base Shear vs Displacement on X Direction

The results from the ETABS provided the parameters needed to calculate the target displacement, δ_T :

T_e	=	0.299 s
S_a	=	1.3468 g
$\mu_{strength}$	=	1.599
V_y	=	2026.0593 kN
W	=	3006.8994 kN
C_m	=	0.8 (Tabel 7-4 ASCE 41-17)
C_0	=	1.3360
C_1	=	1.1123
C_2	=	1.0051
δ_T	=	0.0444 m

The target displacement of the shear wall structure in the X direction is therefore, δT = 0.0444 m.

➢ *Y Direction*

The results from the ETABS analysis provided a pushover capacity curve as shown in Figure 9.

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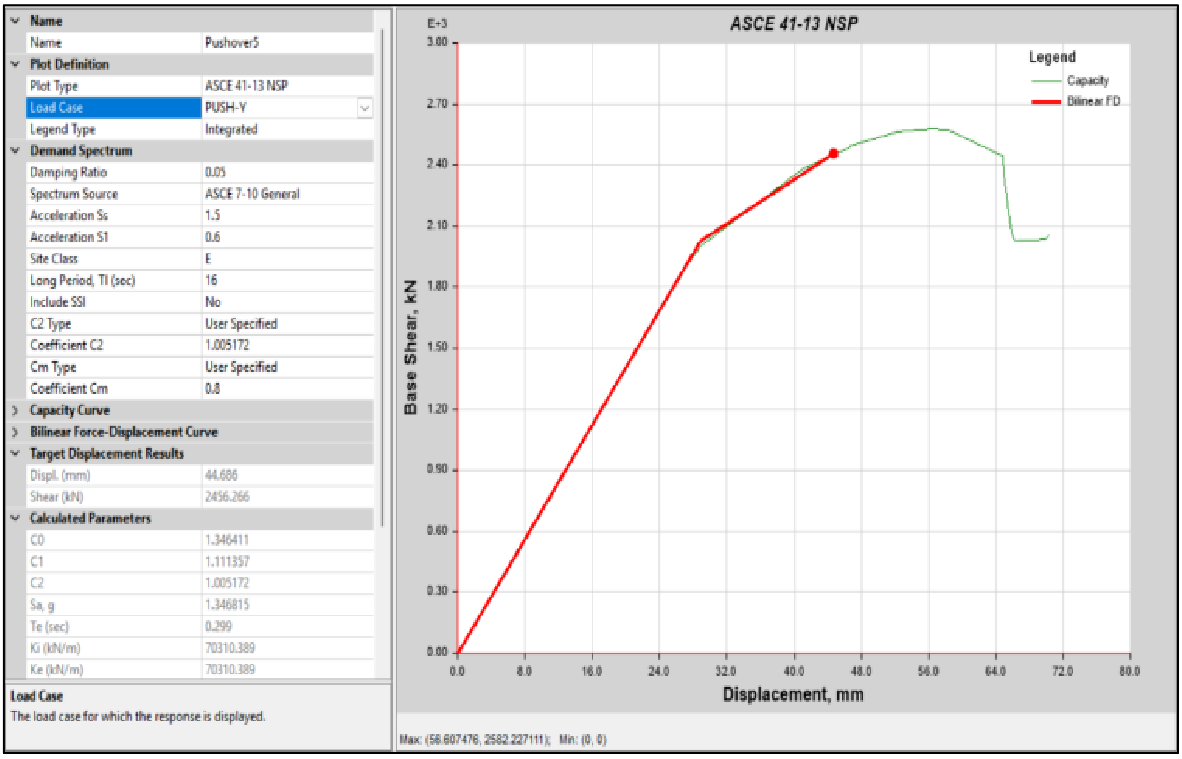

Fig 9 Curve Between Base Shear vs Displacement on Y Direction

The results from the ETABS provided the parameters needed to calculate the target displacement, δ_T :

T_e = 0.299 s
S_a = 1.3468 g
$\mu_{strength}$ = 1.599
V_y = 2026.0593 kN
W = 3006.8994 kN
C_m = 0.8 (Tabel 7-4 ASCE 41-17)
C_0 = 1. 3464
C_1 = 1. 1113
C_2 = 1.0051
δ_T = 0.0446 m

The target displacement of the shear wall structure in the X direction is therefore, δ_T = 0.0446 m.

C. Structural Performance Evaluation

The results of the displacement target calculations are δ_T = 0.0444 m in the X direction and δ_T = 0.0446 m in the Y direction. The maximum allowable deflection for Life Safety (LS) structural performance is 1% of the building's height [8]. Therefore, the maximum deviation is 1.0% × 23.3 m = 0.233 m.

From the results of the pushover analysis in the X direction in Step 7, displacement value of 0.04348 m which is close to the target displacement of δ_T = 0.0444 m, with a base force of 2,432.65 kN. The results are shown in Table 5. From this table, it can be seen that several structural elements have reached the fatigue limit (hinge) and fall into the Life Safety (LS) category based on their target displacement (δ_T).

Table 5 Number of Step ad Displacement of Pushover Analysis in X Direction

Step	Monitored Displ (mm)	Base Force (kN)	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
0	0	0	258	0	0	0	0	258	0	0	0	258
1	10.852	771.41	258	0	0	0	0	258	0	0	0	258
2	21.704	1542.84	258	0	0	0	0	258	0	0	0	258
3	26.997	1905.86	258	0	0	0	0	258	0	0	0	258
4	28.420	1989.92	254	4	0	0	0	258	0	0	0	258
5	40.535	2369.60	254	4	0	0	0	256	2	0	0	258
6	43.248	2420.65	254	4	0	0	0	256	2	0	0	258
7	**43.481**	**2432.65**	**254**	**4**	**0**	**0**	**0**	**256**	**2**	**0**	**0**	**258**
8	53.773	2549.62	254	4	0	0	0	256	2	0	0	258
9	56.060	2547.50	254	4	0	0	0	256	2	0	0	258
10	62.892	2406.42	254	4	0	0	0	256	0	2	0	258
11	73.615	2346.86	254	4	0	0	0	256	0	2	0	258
12	74.265	2351.04	254	2	2	0	0	256	0	2	0	258
13	74.371	2356.70	254	2	2	0	0	256	0	2	0	258

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14	74.372	2356.78	254	2	2	0	0	256	0	2	0	258
15	74.277	2357.04	254	2	2	0	0	256	0	2	0	258

Meanwhile, in the Y direction, as determined in Step 5, the structural displacement value of 0.04108 m is close to the target structural displacement of δ_T = 0.0446 m, with a base force of 2,386.07 kN. The results are shown in Table 6. From this table, it can be seen that fatigue (hinge failure) has occurred in several structural elements that fall under the Life Safety (LS) category based on their target displacements (δ_T).

Table 6 Number of Step ad Displacement of Pushover Analysis in Y Direction

Step	Monitored Displ (mm)	Base Force (kN)	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
0	0	0	258	0	0	0	0	258	0	0	0	258
1	10.852	763.01	258	0	0	0	0	258	0	0	0	258
2	21.704	1526.02	258	0	0	0	0	258	0	0	0	258
3	27.463	1918.13	258	0	0	0	0	258	0	0	0	258
4	28.955	2005.53	254	4	0	0	0	258	0	0	0	258
5	**41.083**	**2386.07**	**254**	**4**	**0**	**0**	**0**	**256**	**2**	**0**	**0**	**258**
6	46.509	2486.52	254	4	0	0	0	256	2	0	0	258
7	46.679	2495.09	254	4	0	0	0	256	2	0	0	258
8	52.358	2565.13	254	4	0	0	0	256	2	0	0	258
9	53.326	2570.74	254	4	0	0	0	256	2	0	0	258
10	56.607	2582.23	254	4	0	0	0	256	2	0	0	258
11	57.607	2576.47	254	4	0	0	0	256	2	0	0	258
12	58.183	2574.78	254	4	0	0	0	256	2	0	0	258
13	64.833	2445.90	254	3	1	0	0	256	1	1	0	258
14	64.963	2356.35	254	3	1	0	0	256	1	1	0	258
15	65.718	2095.64	254	3	1	0	0	256	0	1	1	258
16	66.072	2034.87	254	3	0	1	0	256	0	1	1	258
17	66.486	2027.25	254	3	0	1	0	256	0	1	1	258
18	66.694	2031.67	254	3	0	1	0	256	0	1	1	258
19	68.262	2030.40	254	3	0	1	0	256	0	1	1	258
20	69.896	2038.31	254	2	1	1	0	256	0	1	1	258
21	70.309	2053.04	254	2	1	1	0	256	0	1	1	258
22	70.315	2051.74	254	2	1	1	0	256	0	1	1	258
23	70.316	2051.76	254	2	1	1	0	256	0	1	1	258

IV. CONCLUSION

➢ *Based on the Results of the Research and Discussion, the Following Conclusions can be Drawn:*

- The tower structure with a shear wall system is capable of withstanding lateral forces up to a predetermined displacement target in both the X and Y directions.
- Displacement Targets for the tower structure with a shear wall system using ASCE 41-17 method, the displacement targets (δ_T) in the X direction were calculated to be 0.0444 m and in the Y direction to be 0.0446 m. The maximum allowable displacement limit based on the Life Safety (LS) category is 0.233 m.
- Based on the pushover analysis results, the structural performance level, as evaluated according to ASCE 41-17 for a tower structure with a shear wall system in the X and Y directions, is Life Safety (LS). The structural elements that have reached plasticity (hinges) remain within their performance limits

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