

FINAL PROJECT

**COMPARING THE STRUCTURAL BEHAVIOR OF
THE REINFORCED CONCRETE USING STRONG AXIS
BEAM AND WEAK AXIS BEAM**

**Submitted to the Universitas Islam Indonesia Yogyakarta to Fulfill the
Requirements to Obtain a Bachelor's Degree in Civil Engineering**



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**CIVIL ENGINEERING UNDERGRADUATE PROGRAM
FACULTY OF CIVIL ENGINEERING AND PLANNING
ISLAMIC UNIVERSITY OF INDONESIA
YOGYAKARTA
2025**

PLAGIARISM STATEMENT

I declare that the Final Assignment report I have produced to fulfill the requirements for obtaining a Bachelor's degree in the Civil Engineering Undergraduate Program at the Faculty of Civil Engineering and Planning, Universitas Islam Indonesia, is entirely my own work. The sections of the Final Project report that I have referenced from other sources have been accurately and ethically documented in compliance with the standards and protocols of academic writing. If any portion of this Final Project report is determined to not be my original work or to contain instances of plagiarism, I acknowledge my willingness to face penalties, including the potential revocation of my academic title as governed by the relevant laws and regulations.

Yogyakarta, 26 January 2024

Making the statement,

A handwritten signature in black ink is written over a yellow revenue stamp. The stamp features the Garuda Pancasila emblem, the text 'REPUBLIK INDONESIA', '10000', and 'METERAI TEMPEL'. A unique alphanumeric code '66954ALX384523708' is printed at the bottom of the stamp.

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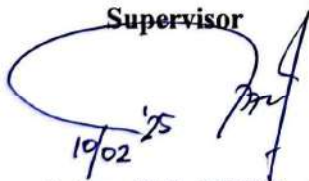
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Has been accepted as one of the requirements
for obtaining a Bachelor's degree in Civil Engineering

Tested on 24 /January/2025
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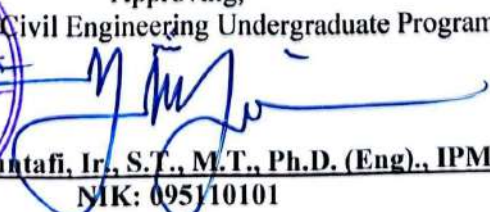
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ABSTRACT

This research is titled "Comparing the Structural Behavior of Reinforced Concrete Using Strong Axis Beam and Weak Axis Beam," using the Kasih Ibu Hospital in Solo, Central Java, as the research object. The building has 13 floors with 6 parking levels, a total height of 53.3 meters, a length of 76 meters, and a width of 30.3 meters. The objective of this research is to determine the changes in structural behavior, beam capacity, and reinforcement requirements due to the change in the beam axis.

The research methodology employed is a comparative method. Two identical building models were created: one with strong-axis beam and the other with weak-axis beam. The structural behavior of these two models was compared to identify differences in structural behavior, beam capacity, and reinforcement requirements due to the axis change.

For the building using weak-axis beam, there was an increase in the building's total weight compared to the building with strong-axis beam, from 255.410,340 kN to 260.174,052 kN. Additionally, the weak-axis beam model exhibited an increase in the building's vibration period compared to the strong-axis beam model, from 1.719 seconds to 1.9015 seconds. This indicates that the structural stiffness of Model 2 is lower than that of Model 1, demonstrating that the structural stability of Model 2 does not meet the requirements of SNI 1726:2019, necessitating the redesign of structural elements to comply with this standard.

The conclusion of this study is that altering the beam axis leads to a reduction in the building's structural stiffness, resulting in various irregularities. Therefore, it is necessary to redesign the structural elements and enhance the reinforcement requirements for beams with altered axes.

Keywords: Beam, Reinforced Concrete, Structural Behavior

FOREWORD

Praise goes to God Almighty for successfully completing this Final Project Proposal entitled " Comparing The Structural Behavior Of *Reinforced Concrete Using Storm Axis Beam And Weak Axis Beam*" .

Purpose of This Final Project is expected can become source information for Who only will take related title with analysis this. This Final Project is one of the academic requirements in completing Bachelor level studies at the Civil Engineering Study Program, Faculty of Civil Engineering and Planning, Indonesian Islamic University, Yogyakarta .

In preparing this Final Project Proposal , there were many obstacles, but thanks to suggestions, criticism, and encouragement from various parties, alhamdulillah, this Final Project Proposal could be completed. In this regard, I would like to express my deepest gratitude to:

1. Mr. Malik Mushthofa , ST, M.Eng as Advisor,
2. Mr. Jafar, S.T., M.T., MURP
3. Mrs. Ir. Yunalia Muntafin ST., MT,. Ph.D.(Eng.), IPM Head of the Civil Engineering Undergraduate Program, Faculty of Civil Engineering and Planning, Indonesian Islamic University,
4. author's parents who have sacrificed both morally and materially,
5. All friends of the Indonesian Islamic University.

Finally, this Final Project is expected to be useful for those who read it.

Yogyakarta, 26 Januari 2025

Writer,



Taufiq Raza Wijaya

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CHAPTER I

INTRODUCTION

1.1 BACKGROUND

From a geographical perspective, Indonesia lies within the trajectory of the Pacific Ring of Fire, an area known for its series of volcanoes. Therefore, it's not surprising that the country experiences frequent seismic activity, including earthquakes, both tectonic and volcanic in nature. According to experts, approximately 81% of significant earthquakes worldwide occur along the Pacific Ring of Fire (Prasetya et al., 2006). Among all natural disasters, earthquakes are often regarded as the most alarming and capable of causing extensive damage. An earthquake is a sudden and rapid shaking of the Earth's surface, typically resulting from the movement and interaction of rock layers beneath the Earth's surface (Veenema, 2007).

As per the Hospital Disaster Plan at RSUD Dr. Moeardi (2020), hospitals play a crucial role in being prepared to treat both emergency and disaster victims. Hence, it is imperative that healthcare institutions, including hospitals, remain undamaged to aid swiftly and accurately those seriously affected by emergencies and disasters. This underscores the necessity for hospitals to be constructed with earthquake-resistant structures, ensuring that they can continue to operate and provide healthcare services during seismic events.

In accordance with Minister of Public Works Regulation No. 29 of (2006) , it is mandated that each non-residential building must allocate a designated area for vehicle parking. This allocation should be proportionate to the building's total floor area. In compliance with the Parking Management Guidelines 272/HK.105/DRJD/96, hospitals have specific requirements for parking space units. These requirements are outlined in Table I.1, Hospital parking space requirements as follows.

Table 1. 1 Hospital parking space requirements

Number of beds (Units)	75	100	150	200	300	400	500	1000
Need (SRP)	100	104	111	118	132	146	160	230

Therefore, as object for this study case the Kasih Ibu Hospital building, which is in the city of Surakarta, Central Java, was chosen as an object studies structure building. The construction process of this building started in the beginning of 2022. In the middle of the construction process, it was realized that the height of the floor used for parking was not enough for emergency vehicles to pass. Then, it was decided to change the design of the beam to become a weak axis beam to increase the clear height of parking area floor. The area of the building where the axis change was made can be seen in figure 1.1 below.

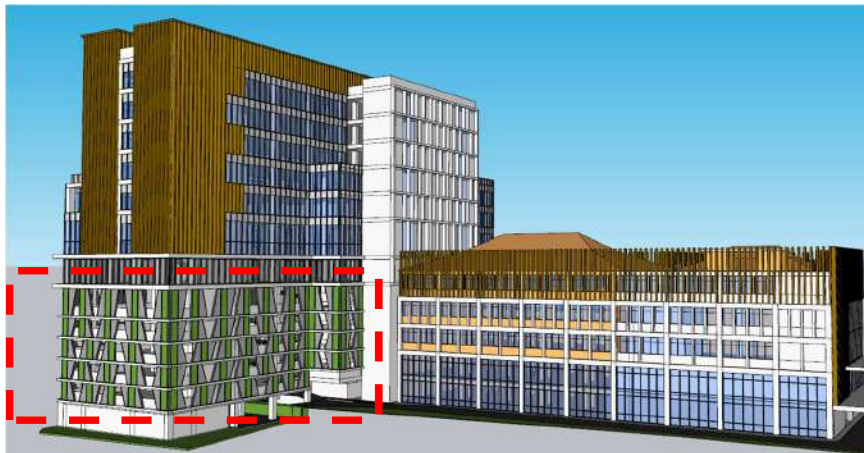


Figure 1. 1 Area of the building where the beam will be rotate

Based on the discussion above, there is a need to conduct an analysis comparing the behavior of the structure due to changes in the axis of the beams. This analysis aims to provide a clearer understanding of how such changes the structural behavior, including the stiffness, P-delta effect, and the resulting reinforcement requirements for the beams.

1.2 Problem statement

From the background explanation above, a problem must be addressed, formulated, and immediately solved.

1. How much is the increase in building weight due to the change in the axis of the beam?
2. How does the change in the axis of the beam affect the stiffness of the building?
3. How does the structural behavior change due to rotation in the axis of the beam?
4. How much is the increase in reinforcement requirements for the beam due to changes in the beam's axis?

1.3 Research Objectives

Based on formula problem above, objective research to be done that is,

1. To know much is the increase in building weight due to the change in the axis of the beam.
2. To find out change in the stiffness of the building due to changes in the beam axis
3. To find out how does the structural behavior changes due to rotation in the axis of the beam.
4. To know how much the increase in reinforcement requirements for the beam is due to changes in the beam's axis.

1.4 Research Benefits

The purpose of this study is to investigate the structural behavior of beams along the strong and weak axis that have an identical beam area. This involves understanding how the structure responds (in terms of displacement and drift) under earthquake loads for both Strong Axis and Weak Axis Beams. Additionally, it aims to determine the necessary reinforcement, assess the differing load-bearing capacities between the two beam types, and compare the associated costs for implementation in hospital projects. Moreover, it is anticipated that this research

will offer valuable insights for future projects involving multi-story buildings with similar structural requirements or in studies of a similar will come.

1.5 Research Limitation

Following the research objectives, so that the scope of the research is clearer and more focused, it is necessary to have a research limitation,

1. Building used as study namely the Kasih Ibu Hospital Building, which is in the city of Surakarta, Central Java.
2. Structural concrete requirements for buildings and explanations using SNI 2847:2019.
3. Planning procedures resilience earthquake for structure buildings and buildings using SNI 1726:2019.
4. Regulation load minimum design and criteria related for building buildings and other structures using SNI 1727:2020.
5. Only conducting analysis on buildings that have parking floors
6. The dimensions of the columns and floor slabs remain unchanged
7. Utilizing a special soil type (Soil Type E) as referenced by the statements provided by PT. Global Rancang Selaras, the planning consultant.
8. Not counting or determining foundation to be used.
9. No cost estimation (Bill of Quantities) calculations were conducted.

CHAPTER II

LITERATURE REVIEW

2.1 Research Studies Previously

In sub chapters this, researchers serve a number of ever research done previously as material for reference study For Comparison of Strong Axis Beams and Weak Axis Beams in Hospitals.

2.1.1 Comparison Analysis of Multi-Story Building Structures with a 12-Meter Span Using SRPMK with Dual System.

This Journal, written by Eka (2017), discusses about comparison of structure behaviour of a 12-meter span beam in a multi-story building with SRPMK, Dual System, SRPMK, and shear walls in x, y, and x/y directions. On research This there are 4 models' structure that is structure that uses Special Moment Resisting Frame System, structure with dual system, structure with dual system and structure with dual systems. The loading and dimension input structure in all modeling is the same. The analysis used in the research is analysis response spectrum based on SNI1726:2012.

Based on analysis that has been done so obtained mark average moment in the beam span 12 meters in Model 1 is 467.67 kN.m, Model 2 is 440.23 kN.m , Model 3 is 607.86 kN.m and Model 4 is 451.62 kN.m.

2.1.2 Analysis of the Behaviour of Irregular Horizontal Buildings.

Pramesti et al. (2018), discusses about analyze behavior building to earthquake load with use dimensional variations column become smaller on the building No order horizontally non- parallel system.

The focus of this study is on Wisma Cipinang Indah, a building characterized by horizontal irregularities in its non-parallel systems. Specifically, it involves the absence of corewalls positioned at the building's center of mass, resulting in slanted corewalls. This structural configuration leads to distinctive responses in terms of

base shift, structural period during shaking, overall displacement, and deviations between levels. The earthquake load analysis is conducted using the dynamic response spectrum approach based on SNI 1726:2012, employing ETABS software version 9.7.4.

The research findings demonstrate that reducing the dimensions of columns in buildings with horizontally irregular, non-parallel systems results in decreased structural stiffness. Consequently, this leads to an amplification of structural style, a shorter period of structural shaking, an increase in base shift style, a reduction in overall displacement, and a decrease in deviations between levels. Additionally, the irregularity in non-parallel systems leads to a significant discrepancy in the average total deviation in the building's behavior along the X and Y axes, amounting to 104.05%. As a result, buildings with varied column dimension reductions and non-parallel system irregularities exhibit reduced resilience to earthquake loads, leading to vastly differing behavior along the X and Y axes.

2.1.3 The Effect of Using weak axis beams on the Seismic Behaviour of Reinforced Concrete Building Structures.

Rahmat (2019), The objective of this study is to analyze the seismic behavior of buildings utilizing weak Axis beams. The subject of the study involves a building of elongated rectangular shape. The investigation encompasses various aspects of the building's structural response, including patterns of base shifting, structural oscillation periods, overall displacement, and deviations between levels. The earthquake load analysis is conducted using dynamic response spectrum analysis, adhering to the guidelines outlined in SNI 1726:2012, and facilitated by the Structural Analysis Robot software version 2019.

The findings indicate that employing weaker Axis beams results in a reduction in structural rigidity, an elongation of structural oscillation periods, a decrease in base shifting, and a reduction in overall displacement as well as deviations between levels. Consequently, opting for wider beams with an equivalent area to standard beams leads to a greater requirement for reinforcement, surpassing that of normal beams by 1.218%.

2.1.4 Design of the Structure of a 5-Story Hospital Building in Semarang City.

Pramanda (2020), purpose Thesis This is designing structure Hospital building 5 floors in accordance with Planning Procedures resilience Earthquake for Structure Buildings and Non-Buildings (SNI 1726:2012). According to SNI earthquake 2012 structure Hospital building must own 1.5 times more power tall compared to buildings other. this because building House Sick must still stand after experience condition extreme, building only can experience damage without experience collapse. Design structure building must in accordance with standards and regulations latest applicable, and with consider style upcoming earthquake happened. because that , the structural design of this 5-storey hospital building designed with refers to Planning Procedures resilience Earthquake For Structure Buildings and Non Buildings (SNI 1726:2012), Structural Concrete Requirements For Buildings (SNI 2847:2013), as well as the Minimum Load for Design Buildings and Other Structures (SNI 1727:2013).In plan A building with height above 30 m, aspect the earthquake load had a big impact planning the . While that can withhold burden earthquake or the resulting lateral loads is structure column from building That itself, basically column function as system crutch something building whereas beams and plates are system link, for That planning interaction column is very important If bang planned can withhold burden earthquake plan.

The Five-Story Building of the Semarang City Type-D Hospital refers tot he regulations of the Department of Public Works, the Guidelines for Development Planning for Homes and Buildings (PPPURG) 1987. The planning of the Five-Story Building of the Semarang City Type-D Hospital includes columns, beams, plates, sloof , Foundation. The location of the building is on Jl. Hadi Soebeno Sosrowardoyo No. 146-148, Wonolopo Village, Mijen District, Semarang City. Using concrete quality ($f'c$) = 30 MPa, for foundations, sloof and beams, using ($f'c$) = 25 MPa for floor slabs, using ($f'c$) = 35 MPa floor slabs for columns, longitudinal reinforcement quality (f_y) = 400MPa and shear reinforcement (f_{yt}) = 240 MPa. software tools used in this plan are Microsoft Office, AutoCad, and SAP2000.

2.1.5 Comparison of the Structural Design of the Building with Square and Round Columns.

This Study, written by Arsemara (2020), aims to determine the structural comparison between square columns and round columns regarding the forces in the column, this is to determine the difference in the amount of reinforcement in square columns and round columns, as well as to find out which columns are more efficient between square columns and round columns. The design begins by dimensioning all structural elements, then loading calculations are carried out using static analysis using SAP 2000 V.14. Comparative analysis was carried out between structures with square columns and round columns.

Based on analysis comparison forces in and amount reinforcement column and round column, square column have forces in more big compared to round column, where obtained style in maximum Axial = 6798.400 kN , Shear = 405.167 kN , Shear = 109.887 kN , Moment = 352.370 kN , Moment = 1348.274 kN , while Round column . Axial = 6391.485 kN , Shear = 157.345 kN , Shear = 79.753 kN , Moment = 260.742 kN , Moment = 509.793 kN , from results analysis style in the with percentage obtained round column smaller 37% of square column . Based on amount reinforcement round / circle column have amount more reinforcement small compared to square column, with percentage reinforcement 53%. Based on comparison wide cross section rounded column = wide cross section column square.

2.2 Summary of Research

The differences in the objectives and the output from previous research are explained in Table 2.1

Tabel 2. 1 Comparison of Research

Aspect	Previous Study					Present study
	Muhammad Eka Kurniawan (2017)	Nadia Rahma Pramesti (2018)	Rahmat Hidayat (2019)	Pramanda Aji (2020)	Lalu Arsemara Hendra Pranata (2020)	Taufiq Raza Wijaya (2024)
Title of Study	Comparison Analysis of Multi-Story Building Structures with a 12-Meter Span Using SRPMK with Dual System.	Analysis of the Behaviour of Irregular Horizontal Buildings.	The Effect of Using weak axis beams on the Seismic Behaviour of Reinforced Concrete Building Structures.	Design of the Structure of a 5-Story Hospital Building in Semarang City.	Comparison of the Structural Design of the Building with Square and Round Columns.	Comparing The Structural Behavior Of The Reinforced Concrete Using Strong Axis Beam And Weak Axis Beam

Continuation of Tabel 2.1 Comparison of Research

Aspect	Previous Study					Present study
	Muhammad Eka Kurniawan (2017)	Nadia Rahma Pramesti (2018)	Rahmat Hidayat (2019)	Pramanda Aji (2020)	Lalu Arsemara Hendra Pranata (2020)	Taufiq Raza Wijaya (2024)
Purpose of Study	The study compares the structure behaviour of a 12-meter span beam in a multi-story building with SRPMK, Dual System, SRPMK, and shear walls in x, y, and x/y directions.	Study This aim for analyze behavior building to earthquake load with use dimensional variations column become more small on the building No order horizontally non-parallel system.	The objective of this study is to analyze the seismic behavior of buildings utilizing weak Axis beams	purpose Thesis This is designing structure Hospital building 5 floors in accordance with Planning Procedures resilience Earthquake for Structure Buildings and Non-Buildings (SNI 1726:2012)	This research compares square and round columns in terms of forces, reinforcement, and efficiency, aiming to identify the most efficient columns.	The purpose of this research is to understand the changes in structural behaviour of the building and the changes in the ultimate moment of the beam after the reversal of the axis from the strong axis to the weak axis.

Continuation of Tabel 2.1 Comparison of Research

Aspect	Previous Study					Present study
	Muhammad Eka Kurniawan (2017)	Nadia Rahma Pramesti (2018)	Rahmat Hidayat (2019)	Pramanda Aji (2020)	Lalu Arsemara Hendra Pranata (2020)	Taufiq Raza Wijaya (2024)
Output of Study	Based on analysis that has been done so obtained mark average moment in the beam span 12 meters in Model 1 is 467.67 kN.m, Model 2 is 440.23 kN.m , Model 3 is 607.86 kN.m and Model 4 is 451.62 kN.m.	Non-parallel systems cause a large 104.05% difference in how buildings behave along the X and Y axes. This irregularity leads to less resilience to earthquake loads, resulting in varied responses in buildings with different column dimensions	Using weak Axis beams makes structures less rigid, increases oscillation times, lowers base shifting, and reduces displacement and level differences. Wider beams of the same area need more reinforcement, exceeding normal beams by 1.218%	understanding the structural calculations of multi-story buildings in Semarang City to comply with SNI standards, prevent building failures, and ensure earthquake resistance	The analysis shows that square columns are stronger than round columns. Overall, round columns are 37% weaker than square columns and have 53% less reinforcement. The cross-sectional width of round columns is equal to that of square columns	Analysis shows that weak axis beams increase the vibration frequency of the building and reduce the structural stiffness, which impacts the deterioration of structural properties. This also reduces the ultimate moment and increases the reinforcement requirements for beams experiencing axis changes.

Continuation of Tabel 2.1 Comparison of Research						
Aspect	Previous Study					Present study
	Muhammad Eka Kurniawan (2017)	Nadia Rahma Pramesti (2018)	Rahmat Hidayat (2019)	Pramanda Aji (2020)	Lalu Arsemara Hendra Pranata (2020)	Taufiq Raza Wijaya (2024)
The Difference with the Present Study	In the present research, beams are used as comparison objects, and only two models are employed, namely the strong axis beam and the weak axis beam.	in the present research, beams are used as the object of study with axis modifications on the main beam	In the present research, not only the structural behaviour analysis, but also conducted the comparison of the increase in reinforcement requirements for beams.	In the present research, we are not redesigning but conducting a comparative analysis of beams.	in the present research, beams are used as the object of study with axis modifications on the main beam	

2.3 Difference Study Previously with Research to be Conducted.

Research that will done use the object of the Surakarta Kasih Ibu Hospital located in Surakarta City, Central Java. In the middle of the construction period, it was realized that the floor height used for parking was not sufficient for vehicles to pass through, so it was decided to change the beam axis from a strong axis to a weak axis. Use Planning Procedures to Resilience Earthquake for Structure Buildings and Buildings using SNI 1726:2019. Study This is different from study before, in this study focuses on changes in structural behavior due to changes in the beam axis and the increase in reinforcement requirements for the beam due to changes in the beam's axis.

CHAPTER III

THEORETICAL BASIS

3.1 General

In completing the comparison of the behavior of reinforced concrete structures using strong axis beams and weak axis beams to meet the strength and feasibility criteria required for a building, this literature review will broadly explain the theory and literature studies. The references used for the structural calculations of the Kasih Ibu Hospital building located in Solo, Central Java are as follows:

1. SNI 2847:2019 on Structural Concrete Requirements for Buildings.
2. SNI 1726:2019 regarding the Guidelines for Earthquake Resistance Planning for Building and Non-Building Structures.
3. SNI 1727:2020 on Minimum Loads for the Design of Buildings and Other Structures.

The load planning on building structures consists of dead loads, live loads, wind loads, seismic loads, and snow loads.

3.2 Reinforced concrete

Reinforced concrete is a composite material widely used in construction due to the combination of the superior properties of concrete and steel reinforcement. Concrete has high compressive strength but is weak in tension, whereas rebar has significant tensile strength. By integrating rebar into concrete, reinforced concrete structures can effectively withstand tensile, compressive, shear, and bending moments. The working principle of reinforced concrete is based on the strong bond between concrete and rebar, allowing these two materials to work synergistically in facing various types of loads.

In the context of construction regulations in Indonesia, SNI 2847:2019 on Structural Concrete Requirements for Buildings establishes technical guidelines for the planning, execution, and quality control of reinforced concrete. This standard

covers important aspects such as the calculation of design strength of structural elements, determination of dimensions and reinforcement, as well as rules regarding reinforcement detailing to ensure optimal structural performance. SNI 2847:2019 also regulates specific provisions related to deflection, cracking, and ductility of structural elements, as well as special requirements for buildings designed to be earthquake resistant.

The main components of reinforced concrete include concrete as a structural material to withstand compressive loads and reinforcing steel that functions to withstand tensile forces. Reinforcement is divided into longitudinal reinforcement as the main load-bearing element and transverse reinforcement (stirrups) that functions to resist shear forces and maintain the stability of the longitudinal reinforcement. The quality of the concrete and reinforcement steel used must meet the established quality standards, including material testing to ensure consistency in strength and durability.

Reinforced concrete has several advantages that make it the primary choice in construction. This material offers high structural strength, durability against corrosive environments, design flexibility for various shapes and sizes of structures, and relatively economical costs. Its applications include various structural elements such as beams, columns, floor slabs, shear walls, and foundations. To ensure the safety and resilience of the structure, especially in earthquake-prone areas, reinforced concrete must be designed with consideration of the seismic design principles regulated in SNI. Thus, the application of reinforced concrete according to these standards can result in buildings that are safe, efficient, and durable.

3.3 Structure

A structure can be composed of several elements, with different properties or characteristics. Based on the constituent elements, the structure can be divided into 4 namely Beam-Column Structures, Trusses Structures, Rigid Frame Structures (Frame Structures), and Shell Structures (including plates, Shell, and Membranes) (SNI 1726: 2019).

According to Schodek (1999 :8) structure formed with method put element horizontal rigidity above element rigid vertical is common structure found. Element horizontally (beam) carry load Which Work in a manner transverse from length and transfer load the to column vertical Which build it. Column the load in a manner axial by beam, then transfer burden That to ground. Columns accept style centralized, generally from ends beam. So, clear There is connection Which tightly between pattern from focus system Which stretched vertical and system pedestal Which stretched horizontally.

According to Schodek (1999:10), flat plate and wall is structure rigid shaper surface. Something wall bearer burden usually can carry good working load in direction vertical nor lateral loads (wind, earthquake, etc.). A flat plate is usually used horizontally and carries load as flex and pass it on to pedestal.

Wall shift is slabs concrete boned Which installed in position vertical on side building certain Which function add stiffness structure and absorb style shift Which big along with the more height structure (Hasanand Astira, 2013). Element structure rigid wall shift or *shear wall* This influential for withhold too lateral force large charge to column. With so, wall shift This will support horizontal styles whereas column only There is carry style normal or style vertical just.

In SNI 1726:2019 it is stated that, Structure building consists of superstructure and Substructure. The superstructure is the part of the building structure that is above ground level. Substructure is part of the building structure that is located below the ground surface, which may consist of a basement structure, and/or the foundation structure. Procedure analysis and design used seismic in planning structure building and its components must own system retainer complete lateral and vertical forces, which are capable of give strength, stiffness, and capacity dissipation enough energy to withstand motion land design in limitations need required deformation and strength.

3.3.1 Superstructure

The components contained in the section structure on building among others as following:

1. Column.

Columns are components that have role important in a building. Function column is successor burden whole building to foundation. Column included structure main for forward heavy buildings and other loads such as burden live.

column must design for withhold style axial from burden factored in all floor or roof and moment maximum from material factored into one span floor or the roof next door under review. Condition the burden that gives ratio moment maximum to burden axial must also be reviewed. on order or construction continuous consideration must give to influence burden floor or no roof balanced good interior and exterior columns as well as from loading eccentric consequence reason other.

2. Beam

Beam is part of working structure for sustain floor above as well as distributor moment to the columns that support it. Supported beam directly on the column called with beam parent, while the one that rests on the main beam called secondary beam. reinforcement double in design beams in general addressed for increase ductility looks, control deflection period long consequence exists crawl and shrink. (McCormac, 2003).

Strong and weak axes are terms used to describe the bending behaviour of beams, particularly in the context of structural engineering. They refer to the two principal axes about which a beam can bend, and their relative strengths in resisting bending.

Strong Axis:

- A. The strong axis of a beam is the axis about which it has the larger moment of inertia (I). This means that it can resist bending about this axis more effectively, with less deflection for a given applied load.

- B. In most common beam shapes, such as I-beams, C-channels, and rectangular sections, the strong axis is the axis that runs parallel to the web of the beam.

Weak Axis:

- A. The weak axis of a beam is the axis about which it has the smaller moment of inertia. This means that it is less resistant to bending about this axis and will deflect more for the same applied load compared to bending about the strong axis.
- B. In the same common beam shapes mentioned above, the weak axis is the axis that runs perpendicular to the web of the beam.

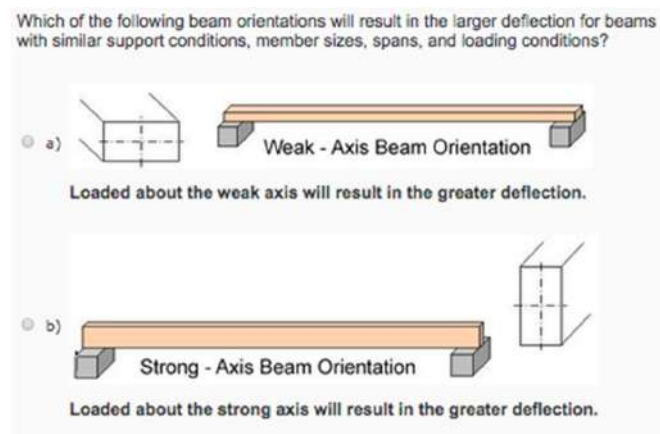


Figure 3. 2 Strong axis and weak axis beam illustration

Procedures for beam component detailing as stipulated in SNI 2847:2019, Article 21. The following are several requirements specified in that article:

- Factored axial compressive force $P_u < A_g \cdot f_c / 10$.
- At each cross-section of a flexural structural component, there shall be at least two continuous top reinforcement bars and two continuous bottom reinforcement bars.
- The positive flexural strength of a flexural structural component at the column face shall not be less than half of its negative flexural strength.

- Both the negative and positive flexural strengths at any section along the span shall not be less than one-quarter of the largest flexural strength provided at either column face.
- The clear span of the structural component l_n shall be greater than four times the effective depth of the beam.
- The width of the component b_w , shall exceed $0.3 h$ or 250 mm.
- The width of the structural component b_w shall not exceed the width of the supporting structural component.
- 0.75 times the overall dimension of the supporting structural component.
- The reinforcement ratio, ρ , shall not exceed 0.025.
- Lap splices for flexural reinforcement are permitted only if stirrups or spirals are provided along the splice length at a spacing of :
 1. $d/4$
 2. 100 mm
- Splices shall not be located within the plastic hinge region.
- The length of the plastic hinge shall be taken as the greater of::
 1. $l_n/4$
 2. $h \times 2$
- The first stirrup shall be placed at 50 mm from the column face.
- The stirrup spacing shall not exceed the following values:
 1. $d/4$
 2. $6 \times D$ tul
 3. 150 mm

Flexural Reinforcement Requirements:

Determining the value of β_1 based on SNI 2847-2019, Article 10.2.7.3

$$\beta_1 = 0,85 - 0,05 \left(\frac{f'c - 7\text{mpa}}{28\text{mpa}} \right) \quad (3-1)$$

$$\rho_b = \frac{0,85 \beta_1 f'c}{f_y} \left(\frac{f'c - 7\text{mpa}}{28\text{mpa}} \right) \quad (3-2)$$

$\rho_{\max} = 0.025$ or $\rho_{\max} = 0.75\rho_b$ (Chose the smallest)

$$\rho_{\min} = \frac{0,25\sqrt{f'c}}{f_y} \quad \text{or} \quad \rho_{\min} = \frac{1,4}{f_y} \quad (\text{Chose the biggest})$$

A_{smin} = Minimum reinforcement area (mm²)

$$A_{smin} = \frac{0,25\sqrt{f'c}}{f_y} b w . d \quad (3-3)$$

Calculate m :

$$m = \frac{f_y}{0,85 f'c} \quad (3-4)$$

Calculate R_n :

$$R_n = \frac{M_n}{\phi b d} \quad (3-5)$$

Determine the required reinforcement ratio:

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2m R_n}{f_y}} \right) \quad (3-6)$$

Area reinforcement need :

$$A_s = \rho b d \quad (3-7)$$

Cek actual Moment nominal (kNm):

$$M_n = A_s f_y \left(d - \frac{a}{2} \right) \quad (3-8)$$

Where :

$$a = \frac{A_s f_y}{0,85 f'c b} \quad (3-9)$$

Shear reinforcement requirements:

According to SNI 2847-2019, Article 21.5.4.1, the design shear force (V_u) must be determined from the static shear analysis at the section of the structural component between two joint faces

$$V_1 = \frac{M_{pr}^- + M_{pr}^+}{0,85 f'c} \pm \frac{W_u L_n}{2} \quad (3-10)$$

Where :

- V_1 = design shear force (kN)
- M_{pr} = flexural strength at the ends of the beam (kNm)
- L_n = length of the beam (m)
- W_u = uniform load (kN)

The load combination used to calculate the design shear force is 1,2D+1L.

Calculate the value of V_s (the shear strength provided by the shear reinforcement)

$$V_s = \frac{V_u}{\phi} - V_c \quad (3-11)$$

Where :

$$\phi = 0.75 \text{ (SNI 2847 – 2019 Article 9.3.2.3)}$$

$$V_c = 0, \text{ If it meets SNI 2847-2019, Article 21.5.4.2}$$

If not, then V_c is calculated as :

$$V_c = 0.17 \times \lambda \times \sqrt{f_c'} \times b_w \times d \quad (3-12)$$

Calculate amount shear reinforcement need :

$$\frac{A_v}{s} = \frac{V_s}{f_y d} \quad (3-13)$$

Where :

- V_s = Shear force resisted by the steel reinforcement.
- A_v = Cross-sectional area of the shear reinforcement.
- S = Spacing between shear reinforcement.
- D = Effective depth of the beam.
- F_y = Yield strength of the steel.

The maximum spacing between shear reinforcements in the support region shall be taken as the smaller of the following :

- $S_{max} \leq \frac{1}{4} d$
- $S_{max} \leq 6$ Multiplying by diameter of the flexural reinforcement.
- $S_{max} \leq 150 \text{ mm}$

3. Floor Slabs

Floor slabs are floors that are not directly above the ground, so they are level floors. This floor plate is supported by beams that rest on building columns.

3.3.2 Construction Substructure

The substructure is part of a building that is located below ground level. The substructure is useful for supporting the structure so that it must have a strong structure, not easily move, and in a stable condition.

3.4 Procedures for Planning Earthquake-Resistant Buildings

3.4.1 Earthquake Zone Map

Based on SNI 1726:2019, Article 14, it explains the seismic ground motion maps and the maximum considered earthquake (MCE) seismic risk coefficient, as shown in Figures 3.1 through 3.4.

Figure 3.1 shows the maximum considered earthquake (MCE) map for targeted risk ground motion parameters SS for site class SE. Figure 3.2 shows the MCE map for targeted risk ground motion parameters S1 for site class SE, Figure 3.3 shows the PGA value for the maximum considered earthquake, represented as the geometric average for site class SE, and Figure 3.4 show Long-period transition map, TL, of Indonesia.

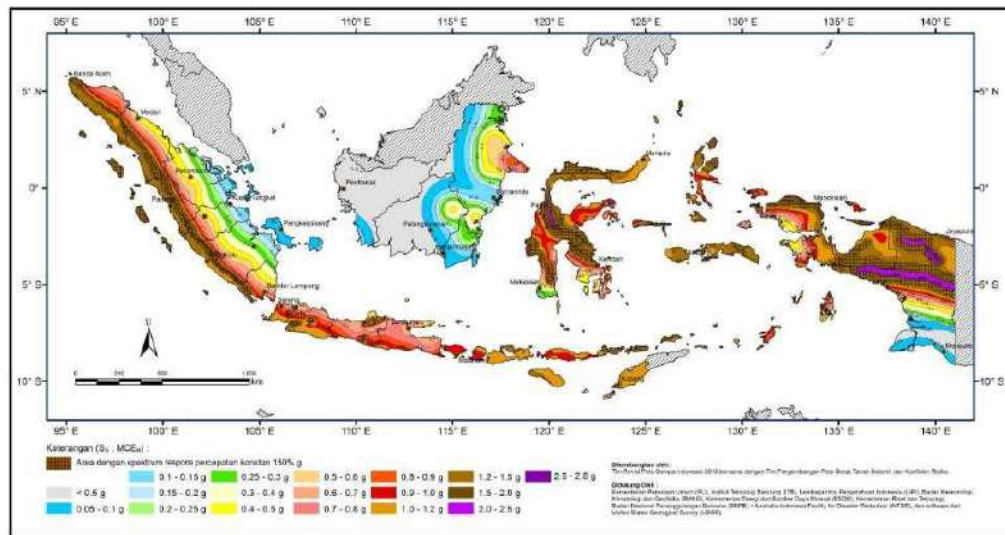
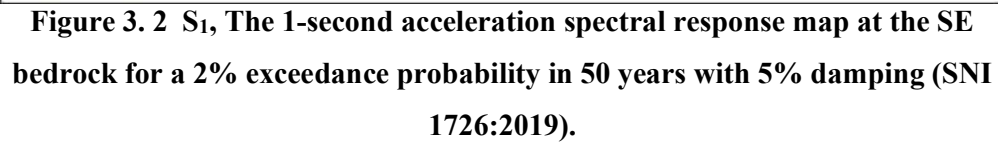


Figure 3. 1 Ss, The 0.2-second acceleration spectral response map at the SE bedrock for a 2% exceedance probability in 50 years with 5% damping (SNI 1726:2019).



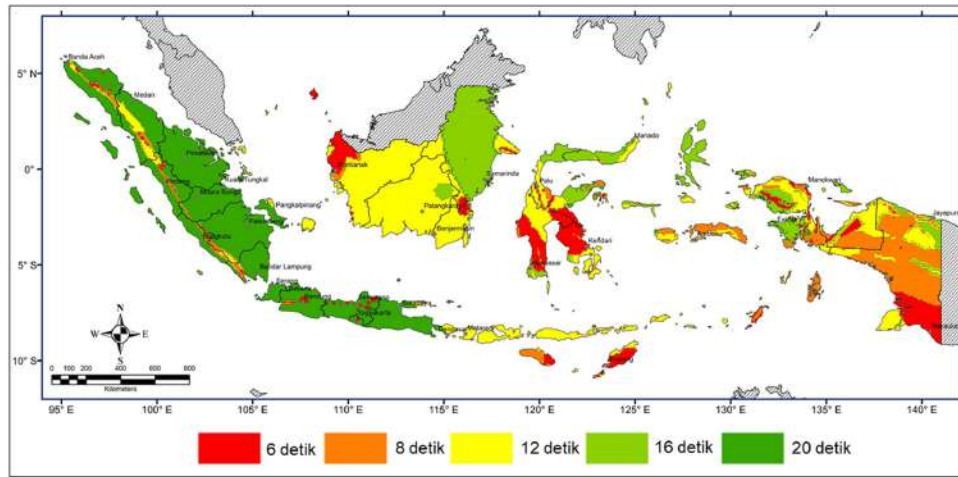


Figure 3. 4 TL, Long Period Transition, Indonesia Region (SNI 1726:2019).

3.4.2 Structural Design Criteria

For planning or designing something structure reinforced concrete require that influence earthquake must plan reviewed in planning and evaluation structure building buildings and non-buildings cert various parts and equipment in a manner general. Earthquake plan set as earthquake with possibility missed magnitude during age structure 50-year-old building is 2 percent. (SNI 1726-2019 article 4.1.1).

The analysis and design procedure used seismic forces, as explained in SNI 1726-2019, Article 7.1.1, states that a building structure must have a complete lateral and vertical force-resisting system, capable of providing sufficient strength, stiffness, and energy dissipation capacity to withstand ground motion. The design must ensure that the structure limits deformation and strength requirements. Ground motion design must account for potential movements in all horizontal directions of the building. The earthquake design type and its distribution along the building's height must be determined based on the appropriate procedures outlined in Table 13 of SNI 1726-2019. The design must also consider the corresponding deformations of the structural components.

3.4.3 Seismic Risk Category

Determine seismic risk category is the very first and crucial step in the design of earthquake-resistant structures. The location of the building affects the magnitude of the seismic forces that the structure will experience. In this process, the following steps are taken:

1. **Geographic Location Identification** Project location identification is carried out by referring to the latitude and longitude coordinates obtained using a geospatial mapping system. This geographical data is used to access earthquake hazard maps that provide information about ground acceleration due to earthquakes (s_a) and identify areas with high earthquake risk.
2. **Earthquake Hazard Map** This map provides important information to determine the peak ground acceleration due to an earthquake at the project location. In SNI 1726-2019, there are two parameters used for the analysis:
 - **S_s**: Response spectrum acceleration for short periods (0,2 seconds), which is relevant for buildings with high natural frequencies such as low-rise buildings.
 - **S₁**: Response spectrum acceleration for long periods (1 second), which is relevant for taller structures or those with long natural periods. This earthquake map provides data used in the calculation of seismic forces for structural design.
3. **Building Risk Level Classification**, each building is categorized based on the importance of its function and the potential impact of damage if the building is damaged due to an earthquake. This classification affects the Earthquake Risk Factor (I_e), which determines the multiplier used in the calculation of earthquake forces.
 - **Low Risk (I)**: Buildings whose damage does not significantly impact public function, such as warehouses or utility buildings.
 - **Medium Risk (II)**: Buildings with general functions, such as offices or residences.
 - **High Risk (III)**: Buildings that play an important role in society, such as cinemas, stadiums, and healthcare facilities without surgical units

and emergency departments.

- Very High Risk (IV): Facilities that must remain operational after an earthquake, such as school buildings and educational facilities, hospitals and other healthcare facilities with surgical and emergency units, and other vital facilities.

3.4.4 Classification of Site Conditions (Soil)

The condition of the soil beneath the structure greatly affects the building's response to earthquakes. Each type of soil has characteristics that affect the magnitude of the seismic waves transmitted and amplified. SNI 1726-2019 divides soil into several categories based on its physical and geotechnical properties:

1. The shear wave velocity (V_s) is used to measure soil stiffness. The faster the waves move through the ground, the stronger the ground is. This affects the level of earthquake amplification received by the structure.
2. Standard penetration test (N-SPT) is used to measure the soil's resistance to the penetration of the testing device. This value is used to identify the density and compaction of the soil and to provide an overview of the soil's strength.
3. Based on V_s and other geotechnical tests, the soil is classified into several categories:
 - A (Hard Rock): $V_s \geq 1500$ m/s. This soil has the ability to transmit seismic waves without amplifying them.
 - B (Moderate Rock): $760 \leq V_s < 1500$ m/s. This soil has moderate strength and can slightly amplify the effects of an earthquake.
 - C (Moderate Soil): $360 \leq V_s < 760$ m/s. This type of soil has the potential to significantly amplify the effects of an earthquake.
 - D (Soft Soil): $V_s < 360$ m/s. This soil amplifies almost all types of seismic vibrations and poses a high risk to building structures.
 - E (Extreme Soil): This soil consists of thick, soft materials such as clay, which is very risky due to high seismic wave amplification.

3.4.5 Structural System

The chosen structural system must be able to respond well to seismic forces, withstand earthquake lateral loads without failure. Several types of structural systems recommended in SNI 1726-2019 are:

1. **Moment-Resisting Frame System (SRPM)**

This system is designed so that its frame elements can rotate (experience moments) and distribute seismic forces. SRPM is suitable for tall buildings or structures that require high flexibility. This system has good energy dissipation capacity, allowing the structure to withstand seismic loads without significant damage.

2. **Shear Wall System**

The shear wall system uses reinforced concrete or steel walls to resist lateral forces. These walls are very rigid and efficient in resisting horizontal loads, making them suitable for medium or tall buildings with the necessary stiffness to withstand earthquakes.

3. **Dual System**

This system combines the two previous systems (SRPM and shear wall). By using a dual system, the structure's capacity to withstand seismic loads increases due to the redundancy provided by both systems. The dual system provides a balance between flexibility and rigidity, as well as redundancy in responding to earthquakes.

3.5 Loading and Combinations

Buildings and other structures shall be designed using the factored load combination provisions used in the strength design method or nominal load combinations using the allowable stress design.

3.5.1 Loading

Load according to SNI 1727: 2013 is a force or other action obtained from the weight of all building materials, occupants, goods inside the building,

environmental effects, displacement differences, and restraint forces due to changes in dimensions.

Structure A building must plan strength to load - burden following:

1. Vertical load (Gravity)

a. Dead load (DL)

Dead load is the total weight of all construction materials installed in a building, including walls, floors, roof, ceiling, stairs, fixed partition walls, finishes, building cladding, and other architectural and structural components, as well as other installed service equipment including the weight of taps (SNI 1727-2013). Dead load can be expressed as a static force caused by the weight of each element within the structure. The forces that contribute to the dead load consist of the weight of load-bearing elements of the building, floors, ceiling finishes, fixed partition walls, facade finishes, storage tanks, mechanical distribution systems, and so on. The combination of the loads from all these elements constitutes the dead load of a building (Schueller, 1989:8). In the analysis and modeling using SAP2000 software, dead loading can be directly calculated.

b. Live load (LL)

Live loads are loads caused by users and occupants of buildings or other structures that are not included in construction loads and environmental loads, such as wind loads, rain loads, earthquake loads, flood loads or dead loads.

2. Horizontal Load (Laterally)

a. Earthquakes load (E)

Earthquake loads are loads that arise due to the acceleration of ground vibrations when an earthquake occurs. The factors that affect the magnitude of the earthquake load that occurs on the building structure, namely the mass and stiffness of the structure, natural vibration time and the effect of damping on the structure, soil conditions, and the seismic area

where the building structure is located. Basically, there are two methods of Earthquake Planning Analysis, namely:

1) Statically Equivalent Analysis

This analysis is a method of structural analysis, in which the effect of an earthquake on a structure is considered as a horizontal static load to mimic the effect of a real earthquake due to ground motion. This method is used for regular structures with a height of not more than 40 m.

2) Dynamic Analysis.

This method is used for building with structure that is not order. Calculation earthquake with analysis dynamic This consists of:

a) Variety Spectrum Response Analysis.

Response Spectrum Analysis is something method analysis dynamic structure. Where something model from math structure enforced something spectrum response earthquake plan and determined response structure to earthquake plan the.

b) Response Spectrum Analysis.

Time History Response Analysis is something method analysis dynamic structure, where something model math from structure-imposed history time from earthquakes results recording or earthquakes imitation to history time from response structure determined.

To determine the spectral acceleration response parameters at the ground surface, amplification factors related to the spectral acceleration for short periods (F_a) and a 1.0-second period (F_v) are required. The spectral acceleration response parameters at the ground surface can then be obtained by multiplying the F_a and F_v coefficients with the spectral acceleration for the short period (S_s) and the 1.0-second period (S_1) at the bedrock, as obtained from

the Indonesia earthquake maps in SNI 1726:2019, according to Equations 3-14 and 3-15.

$$S_{MS} = F_a S_s \quad (3-14)$$

$$S_{M1} = F_v \cdot S_1 \quad (3-15)$$

Where :

S_s = Spectral acceleration value for a 0.2-second short period at the bedrock (SB), referring to the Earthquake Map in SNI 1726:2019 (Figure 3.2).

S_1 = Spectral acceleration value for a 1.0-second period at the bedrock (SB), referring to the Earthquake Map in SNI 1726:2019 (Figure 3.3)

F_a = Short-period coefficient for 0.2 seconds (Table 2.7)

F_v = Period coefficient for 1.0 second (Table 2.8).

Tabel 3. 1 The short-period coefficient, F_a , based on SNI 1726:2012.

Site Class	The spectral response parameter MCER is mapped for the short period, $T = 0.2$ seconds S_s				
Site Classification	$S_s \leq 0.25$	$S_s = 0.5$	$S_s = 0.75$	$S_s = 1.0$	$S_s \geq 1.25$
Hard Rock (S_A)	0.8	0.8	0.8	0.8	0.8
Rock (S_B)	1.0	1.0	1.0	1.0	1.0
Very Dense Soil and Soft Rock (S_C)	1.2	1.2	1.1	1.0	1.0
Medium Soil (S_D)	1.6	1.4	1.2	1.1	1.0
Soft Soil (S_E)	2.5	1.7	1.2	0.9	0.9
Special Soil (S_F)	SS ^b				

Tabel 3. 2 The 1.0-second period coefficient, F_v , based on SNI 1726:2012.

Site Classification (According to Table 2.10)	The spectral response parameter MCER is mapped for the 1.0-second period, S_I				
Site Classification	$S_I \leq 0.1$	$S_I = 0.2$	$S_I = 0.3$	$S_I = 0.4$	$S_I \geq 0.5$
Hard Rock (SA)	0.8	0.8	0.8	0.8	0.8
Rock (SB)	1.0	1.0	1.0	1.0	1.0
Very Dense Soil and Soft Rock (SC)	1.7	1.6	1.5	1.4	1.3
Medium Soil (SD)	2.4	2.0	1.8	1.6	1.5
Soft Soil (SE)	3.5	3.2	2.8	2.4	2.4
Special Soil (SF)	SS ^b				

Next, to obtain the design spectral response parameters, the design acceleration spectrum for the short period, SDS , and for the 1.0-second period, $SD1$, can be obtained through Equations 3-16 and 3-17 as follows:

$$S_{DS} = \frac{2}{3} - S_{MS} \quad (3-16)$$

$$S_{D1} = \frac{2}{3} - S_{M1} \quad (3-17)$$

Where :

S_{DS} = Design spectral acceleration response parameter for the short period.

S_{D1} = Design spectral acceleration response parameter for the 1.0-second period

Next, the design spectral response at the ground surface can be determined as shown in Figure 3.4:

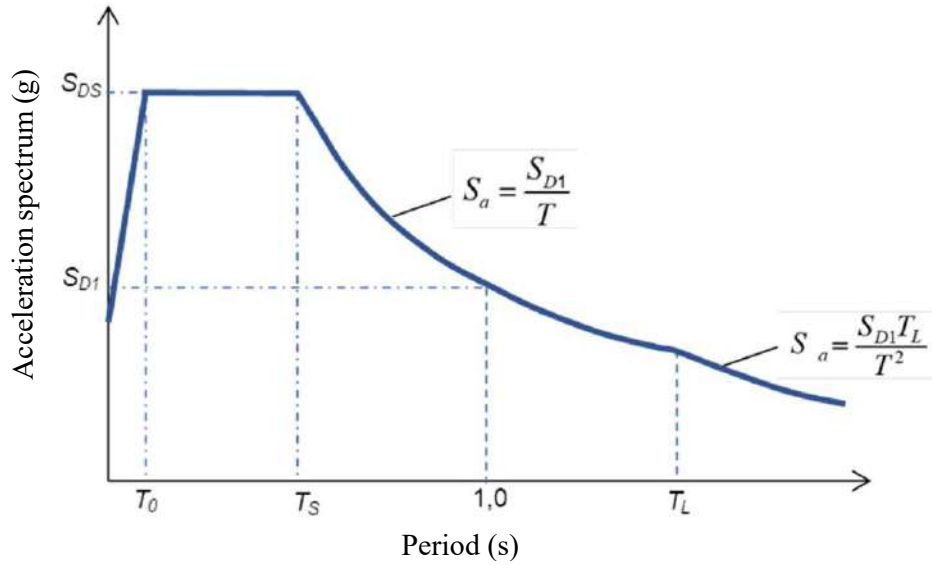


Figure 3. 5 Typical form of the design spectral response at the ground surface (SNI Earthquake: 1726:2019).

Where :

1. For periods smaller than T_0 , the design spectral acceleration response, S_a should be taken from Equation 3-18 as follows :

$$S_a = S_{DS} \left(0,4 + 0,6 \frac{T}{T_0} \right) \quad (3-18)$$

2. For periods greater than or equal to T_0 , and less than or equal to T_s , the spectral acceleration response, S_a is equal to S_{DS} .
3. For periods greater than T_s , the spectral acceleration response, S_a is obtained from Equation 3-19 as follows:

$$S_a = \frac{S_{DS}}{T} \quad (3-19)$$

The values of T_0 and T_s can be determined using Equations 3-20 and 3-21 below:

$$T_0 = 0,2 - T_s \quad (3-20)$$

$$T_s = \frac{S_{D1}}{S_{DS}} \quad (3-21)$$

3.5.2 Load Combination

Structures, components, and foundations must design such shape so that strength the design The same or exceed effect from load factored in combination following: (SNI 03-1726-2019).

1. $1.4DL$
2. $1,2 DL + 1.6LL + 0.5 (Lr \text{ or } R)$
3. $1.2DL + 1.0E + LL$
4. $0.9DL + 1.0 E$

DL = Dead Load (*Dead load*)

LL = Live Load (*Live load*)

Lr = live load on roof (*roof live load*)

E = Earthquake load (*Earthquakes load*)

According to SNI 1727-2013, combination base loading, as following:

1. $1,4D$
2. $1,2D + 1.6L + 0.5 (L_r \text{ or } S \text{ or } R)$
3. $1,2D + 1.6 (L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$
4. $1.2D + 1.0W + L + 0.5 (L_r \text{ or } S \text{ or } R)$
5. $1,2D + 1.0E + L + 0.2S$
6. $0.9D + 1.0W$
7. $0.9D + 1.0E$

3.6 Structure Behavior

The structural properties of a building refer to the characteristics and abilities of the building structure to bear loads and maintain its stability. structure behavior of building structures that can be influenced by loads such as earthquakes and wind. These properties are fundamental in the design and construction of safe and reliable buildings. Structure behavior can be categorized as follows.

A. Strength

Strength is the ability of a structure to withstand loads without experiencing damage. Factors that influence it: Materials with high strength such as steel and reinforced concrete are able to withstand larger loads, a proper and efficient structural design can increase its strength, good construction quality ensures that the structure is built according to the design and is able to bear loads.

B. Stiffness

Stiffness is the ability of a structure to resist deformation due to loads. Factors that influence stiffness is, Materials with high elastic modulus, such as steel, are stiffer than materials with low elastic modulus, such as wood. Proper shape and dimensions of a structure can increase its stiffness and solid shapes like triangles are stiffer than complex shapes.

C. Irregularity

Based on SNI 1726:2019, which discusses the Earthquake Resistance Planning Requirements for Building and Non-Building Structures, irregularity refers to a condition where a structure deviates from its ideal form or configuration, either vertically or horizontally. This irregularity can affect the building's response to seismic loads and increase the risk of structural damage. SNI 1726:2019 divides irregularities into two main categories, namely:

1. Horizontal Irregularities

Horizontal irregularity is a deviation that occurs in the horizontal plane (building floor plan). Types of horizontal irregularities include:

- Torsional Irregularity: If the floor displacement due to an earthquake has a significant difference between the center of mass and the center of rigidity. With the criteria If the maximum lateral displacement on one side of the building is more than 1.2 times the average lateral displacement on both sides. Buildings with torsional irregularities are prone to local collapse due to unanticipated torsion.

- Excessive torsional irregularity is defined to exist if the maximum inter-story drift calculated, including the effect of unexpected torsion with $A_x = 1.0$, at one end of the structure transverse to an axis is more than 1.4 times the average inter-story drift at both ends of the structure. The requirement for excessive torsional irregularity in the reference articles applies only to structures where the diaphragm is rigid.
- Internal corner irregularity is defined to exist if both dimensions of the structural plan projection from the internal corner location are greater than 15% of the structural plan dimensions in the direction being considered.
- The irregularity of diaphragm discontinuities is defined to exist if there is a diaphragm that has discontinuities or sudden stiffness variations, including those with cut or open areas greater than 50% of the gross closed diaphragm area, or changes in effective diaphragm stiffness of more than 50% from one level to the next.
- Irregularities due to vertical shifts against the plane are defined to exist if there is a discontinuity in the lateral force resistance path, such as a vertical shift against the plane in at least one vertical element supporting the lateral force.
- The non-parallel system irregularity is defined to exist if the vertical elements of the lateral load-bearing system are not parallel to the main orthogonal axes of the seismic load-bearing system.

2. Vertical Irregularities

Vertical irregularities occur due to deviations in mass distribution, stiffness, or strength at the height of the building. Types of vertical irregularities include:

- Irregularity Soft Story: defined as present if there is a level whose lateral stiffness is less than 70% of the lateral stiffness of the level above it or less than 80% of the average lateral stiffness of the three levels above it.

- Irregularity Soft Story Stiffness Excessive is defined as the presence of a level whose lateral stiffness is less than 60% of the lateral stiffness of the level above it or less than 70% of the average lateral stiffness of the three levels above it.
- Weight (Mass) Irregularity is defined to exist if the effective mass at a certain level of the item is more than 150% of the effective mass at the nearby level. A roof that is lighter than the floor below it does not need to be reviewed.
- Vertical Geometric Irregularity is defined to exist if the horizontal dimension of the seismic force-resisting system at any level is more than 130% of the horizontal dimension of the seismic force-resisting system at the adjacent level.
- Irregularity Due to Field Discontinuity in Vertical Elements Supporting Lateral Forces is defined to exist if the shift in the direction of the field of the lateral force-supporting element is greater than the length of that element or if there is a reduction in the stiffness of the supporting element at the level below it.
- Weak Story Irregularity Due to Discontinuity in Lateral Strength is defined to exist if the lateral strength of a story is less than 80% of the lateral strength of the story above it. The lateral strength of a level is the total strength of all seismic-resisting elements that share the level shear in the considered direction.
- Excessive Weak Story Irregularity Due to Discontinuity in Story Lateral Strength is defined to exist if the lateral strength of a story is less than 65% of the lateral strength of the story above it. The lateral strength of a level is the total strength of all seismic-resisting elements that share the level shear in the considered direction.

D. Story drift and drift Ratio

Story drift, or inter-story drift, is an important measure of a building's performance when subjected to loads such as earthquakes and wind. Excessive story drift can lead to structural damage such as racks in walls,

connections, and columns damage to other structural elements such as beams and floor slabs in extreme cases, structural collapse. The maximum story drift limit in buildings is generally regulated in building codes in each country. In Indonesia, the maximum story drift limit is regulated in SNI 1726:2019 concerning the Procedure for Planning the Earthquake Resistance of Building Structures.

The value of story drift in a building is influenced by several factors, including:

1. Strength and stiffness of the structure: A strong and stiff structure will have a smaller story drift.
2. Magnitude of lateral loads: The larger the lateral loads, the larger the story drift that occurs.
3. Mass and stiffness distribution: Uneven distribution of mass and stiffness can cause larger story drift in certain parts of the structure.
4. Building shape and geometry: Complex building shapes and geometry can cause larger story drift.

Drift ratio, or the ratio of displacement between floors, is an important parameter for measuring the performance of a building structure when subjected to lateral loads such as earthquakes and wind. Drift ratio is calculated by dividing the story drift by the floor height:

Drift ratio = Story drift average / Drift max.

Story drift is the difference in horizontal displacement between floors in a building due to lateral loads. Floor height is the distance between floors measured. Maximum Limits and Factors Influencing:

The maximum drift ratio for buildings is generally regulated in building codes in each country. In Indonesia, the maximum drift ratio is regulated in SNI 1726:2012 on the Procedure for Planning the Earthquake Resistance of Building Structures. Below is a table of maximum drift ratio limits based on earthquake risk categories:

E. P-Delta

P-Delta, or the P-Delta effect, is a phenomenon that occurs in tall building structures where the vertical force (P) acting on the structure produces bending moment (M) due to lateral displacement (Δ). This moment can cause the structure to experience larger deflections, increasing the risk of damage and collapse.

Mathematically, P-Delta is calculated by the formula:

$$M = P * \Delta \quad (3-22)$$

Where:

M is the bending moment due to the P-Delta effect

P is the vertical force acting on the structure

Δ is the lateral displacement of the structure

Impacts and Risks:

The P-Delta effect can have several impacts and risks on tall building structures, including:

- Increased deflection: Excessive structural deflection can cause discomfort for occupants, cracks in walls and other structural elements, and even structural collapse.
- Increased internal forces: The P-Delta effect can increase internal forces in the structure, such as shear forces and bending moments, which can cause damage to structural elements.
- Instability: The P-Delta effect can cause the structure to become unstable, increasing the risk of structural collapse.

Factors Influencing:

The magnitude of the P-Delta effect on tall building structures is influenced by several factors, including:

- Building height: The taller the building, the greater the P-Delta effect.
- Lateral stiffness: The stiffer the lateral structure, the smaller the P-Delta effect.

- Mass distribution: Uneven mass distribution can increase the P-Delta effect on the structure.
- Lateral loads: The larger the lateral loads acting on the structure, the greater the P-Delta effect.

3.7 Serviceability

The structure must be capable of carrying the design loads safely, without overstressing the material and having deformation limits within permissible limits. Serviceability includes Criteria for strength, namely the selection of dimensions and shapes of structural elements at a level that is considered safe so that excess stress on the material (eg indicated flatness) does not occur. variation in structural stiffness which serves to control the deformation caused by the load. Movement in a structure is also related to deformation. The actual speed and acceleration of structures carrying dynamic loads can be felt by building users and can cause discomfort. In tall building structures there is structural movement due to wind loads. For this reason, criteria regarding speed limits and permitted acceleration are needed. Control will be achieved through manipulation of structural stiffness and damping characteristics.

3.8 Efficiency

Efficiency criteria include the objective of designing structures that are relatively more economical. The indicator that is often used in this criterion is the amount of material needed to carry the load. Each structural system may require different materials to provide the same structural serviceability. The use of minimum volume as a criterion is an important concept for both architects and structural planners.

3.9 Construction

The construction review will also influence structural choices. A construction will be efficient if the material is easily obtained and assembled. The construction criteria are very broad including a review of the means or methods to carry out the building structure, as well as the types and tools required and the time of

completion. In general, assembling with parts whose shapes and sizes are easy to do with existing construction equipment is desirable.

3.10 Economical

Price is factor Which determine election structure. Draft pricerelated with efficiency materials and facilities its implementation. Total price something structure very depend on Lots And price material Which used,as well as cost power Work executor construction, as well as cost equipment Which needed during implementation job.

3.11 Others

Besides factor Which can be measured like criteria before, variouscriteria relatively which is more subjective Also will determine election structure.

CHAPTER IV

RESEARCH METHODS

4.1 Research Data and Literature Studies

Kasih Ibu Hospital in Surakarta is one of the health facilities in the city of Solo, Central Java, which was built to provide health services for the community in the city of Surakarta and its surroundings.

The construction of this building structure started in the middle of the construction period; it was realized that the floor height used for the vehicle parking was not sufficient for vehicles to pass. So, it was decided to change the beam design to a weak axis beam on the floor used for the parking floor. This is the basis for carrying out a comparative analysis of structural behavior due to changes in the axis of the beam.

Literature study refers to some regulations. The following are rules used in study following.

- a. Structural Concrete Requirements for Buildings and Explanations using SNI 2847:2019.
- b. Planning Procedures resilience Earthquake for Structure Buildings and Buildings using SNI 1726:2019.
- c. Regulation minimum load design and criteria related for building buildings and other structures using SNI 1727:2020.

In addition, previous studies similar to this research and books related to this research are also additional reference materials.

4.2 . Research Locations

Research location located on Jl. Slamet Riyadi No. 404, Purwosari , Kec . Laweyan , Surakarta City, Central Java 57142. The research location can be seen on figure 4.1.



Figure 4. 1 Location of Rumah Kasih Ibu Surakarta

following is cross section strong axis beams used. Can be seen in figure 4.2, and the complete details can be found in appendix 1.

KODE	B1-A		B1-B		B2-A	
POSISI	TUMPUAN	LAPANGAN	TUMPUAN	LAPANGAN	TUMPUAN	LAPANGAN
POTONGAN						
DIMENSI	400 X 800	400 X 800	400 X 800	400 X 800	350 X 650	350 X 650
TULANGAN ATAS	13 D 22	3 D 22	10 D 22	3 D 22	8 D 22	3 D 22
TULANGAN BAWAH	6 D 22	6 D 22	5 D 22	5 D 22	4 D 22	5 D 22
TULANGAN BADAN	4 D 10	4 D 10	4 D 10	4 D 10	4 D 10	4 D 10
SENGKANG	4D10-150	2D10-200	4D10-150	2D10-200	2D10-150	2D10-200

Figure 4. 2 Cross section of seams B1-A, B1-B, and B2 -A



Figure 4. 3 Floor plan of the area where the beam will rotate.

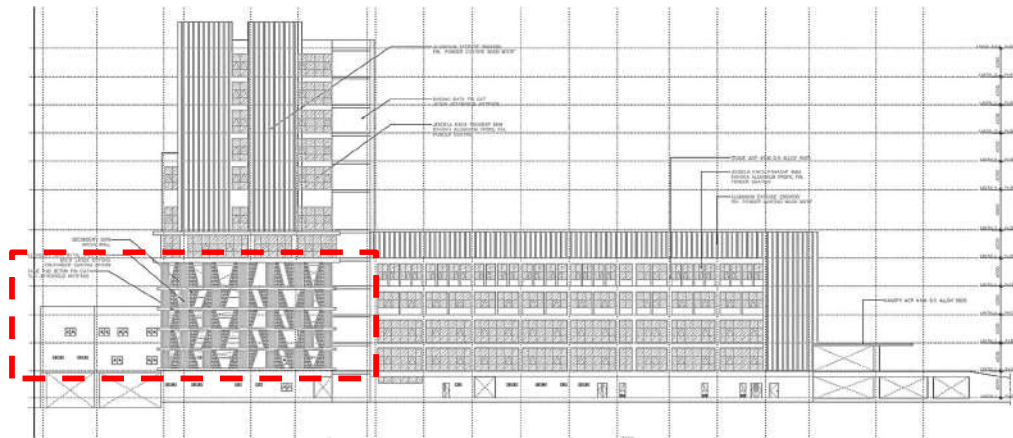


Figure 4. 4 Side view of the area where the beam will rotated.

4.3 Research Flowchart

The steps for implementing the activities carried out in this final assignment can be described through the following flowchart:

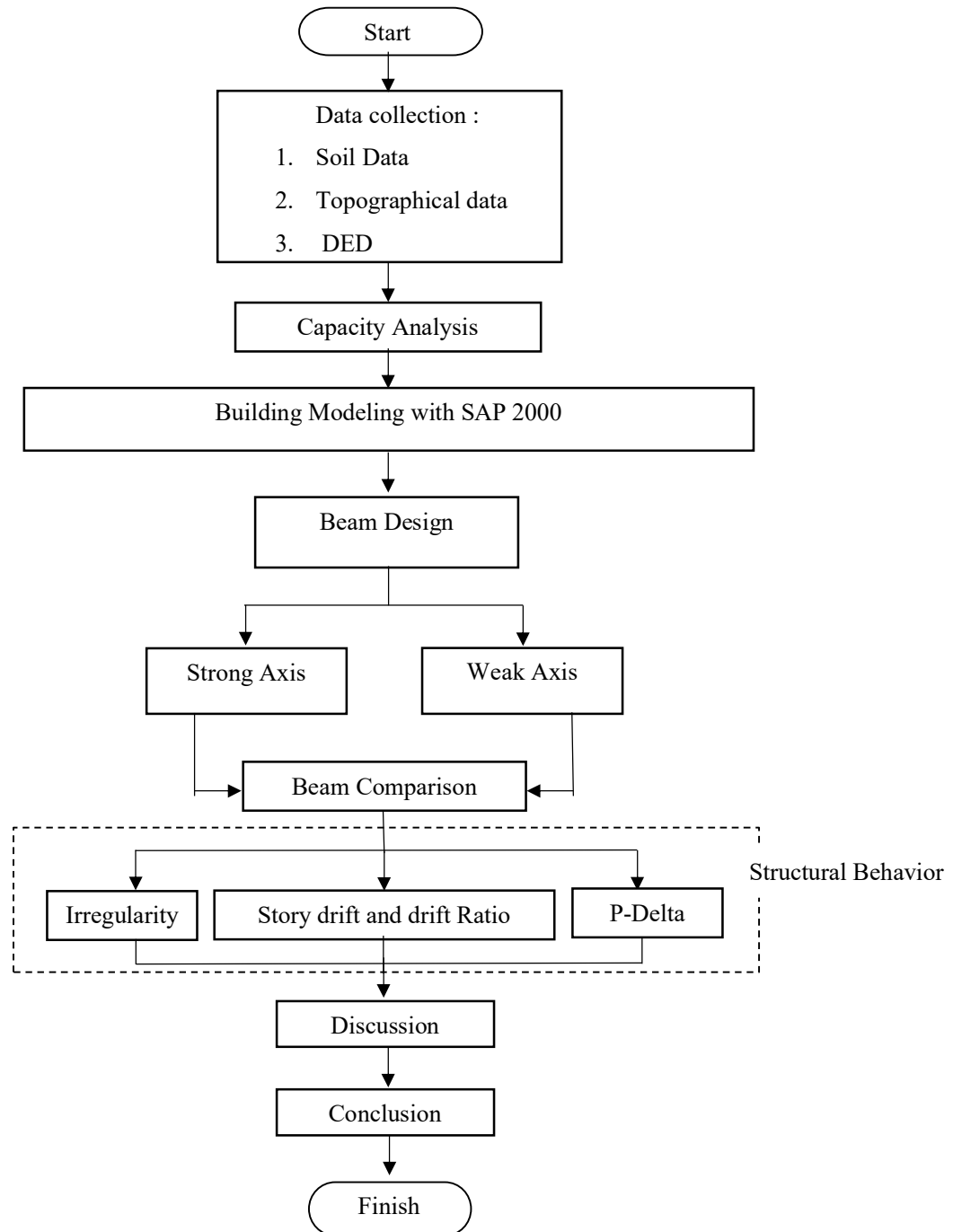


Figure 4. 5 Research flowchart

4.4 Overview

There are 2 structural models in this final project, namely a building structure with a strong axis beam using SRPMK (Model 1) and a building structure with a weak axis beam using SRPMK (Model 2), which in each modelling are only differentiated based on the axis of the main beam in the parking area that is rotated 90°. Both building models will be analysed elastically linearly using response spectrum analysis.

4.5 Structural Planning

4.5.1 General Planning Data

The general planning data used in both structural models are:

1. The planned building structure is a regular building structure with 13 floors, functioning as a parking lot from the Basement to P7. Meanwhile, the other floors are designated as a hospital.
2. The total height of the building is 52 m, with the height of each parking floor is 1,5m and the height of other floors is 4,2m
3. Building used as study namely the Kasih Ibu Hospital Building.
4. The building is located in the city of Surakarta, Central Java.
5. Uses Special Moment Resisting Frame System (SMRFS).
6. Risk Category IV.
7. Category Desain Seismic D (KDS D)

4.5.2 Dimensions of Column and Beams

The planning of the dimensions of columns and beams used in the building structure can be seen in Tables 4.1 and 4.2.

Table 4. 1 Beam Dimension

Beam	Cross-sectional Dimensions		Support Area		Span Area	
	B	H	Top reinf	Bottom reinf	Bottom reinf	Top reinf
B1-A	400	800	13D22	6D22	6D22	4D22
B2-A	350	650	8D22	4D22	5D22	3D22
B2-c	350	650	6D22	4D22	5D22	3D22
B3	300	600	10D19	4D19	5D22	4D19

Continued Table 4.1 Beam Dimension

Beam	Cross-sectional Dimensions		Support Area		Span Area	
	B	H	Top reinf	Bottom reinf	Bottom reinf	Top reinf
B4	300	500	6D19	4D22	3D19	3D19
B5	250	500	9D16	4D16	8D16	6D16
B6	200	400	5D16	2D16	8D16	4D16
T1	400	800	10D22	10D22	10D22	10D22
T2	350	650	6D22	6D22	6D22	6D22
T3	300	600	6D19	6D19	6D19	6D19
T4	300	500	4D19	4D19	4D19	4D19
T5	250	500	6D16	6D16	6D16	6D16
T6	200	400	3D16	3D16	3D16	3D16

Table 4. 2 Column Dimension

Column	B	H	Reinforcement	Shear Reinf
K1	1200	1200	56D22	4D10-150
K2	1000	1000	36D22	3D10-100
K3	900	900	28D22	3D10-100
K4	800	800	20D22	4D10-150
K5	450	1000	40D22	4D10-150
K6	200	1200	16D16	4D10-150
K7	300	300	8D13	2P8-150
K8	200	200	8P12	2P6-180

The location and position of each beam and column on each floor can be seen in Figures 4.3, for more complete images, please refer to appendix 2 to 19.

4.5.3 Floor Slabs Planning

In this final project, there are 4 types of floor slabs used in the structural planning. The first-floor slab design is a reinforced concrete slab with a concrete compressive strength of $f_c' = 30$ MPa and BJTD-42 steel quality. The detailing of the floor slab reinforcement can be seen in figures 4.4 and 4.5.

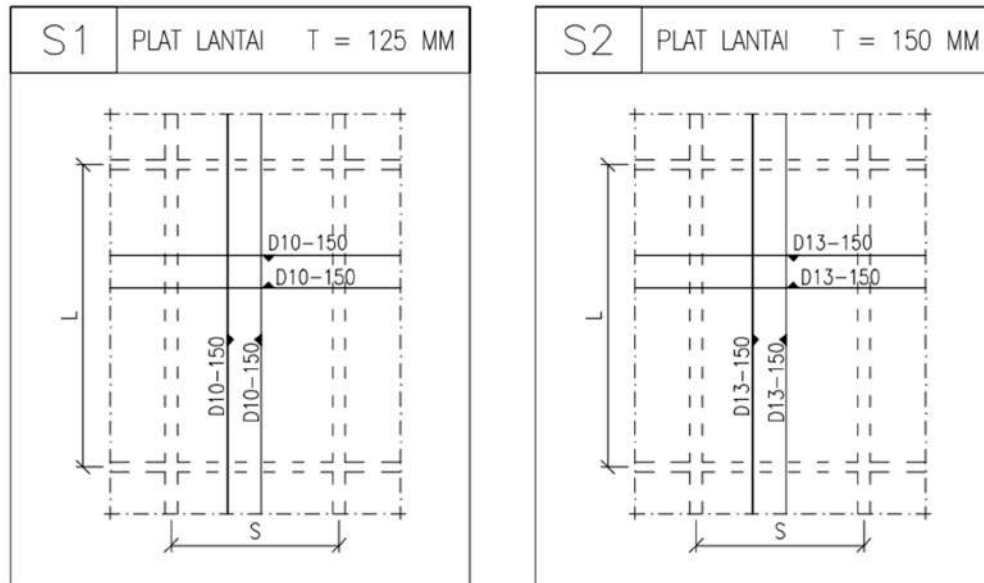


Figure 4. 4 Detail reinforcement of S1 and S2 floor slabs.

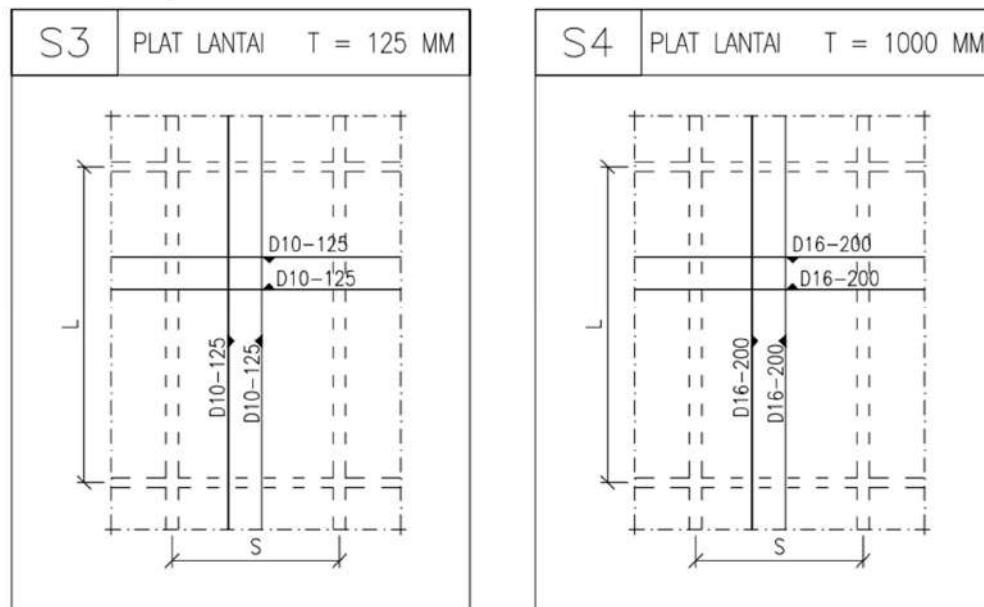


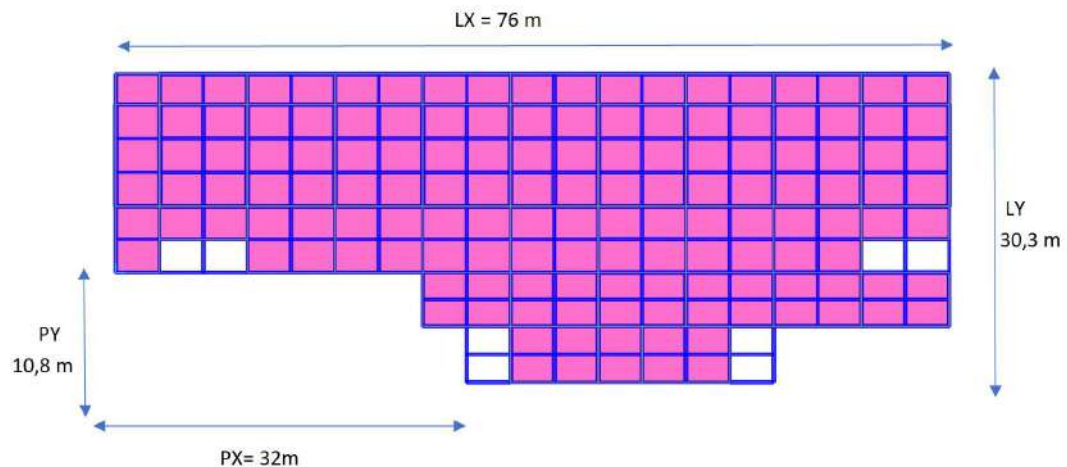
Figure 4. 5 Detail reinforcement of floor plates S3 and S4.

4.6 Modeling Planning

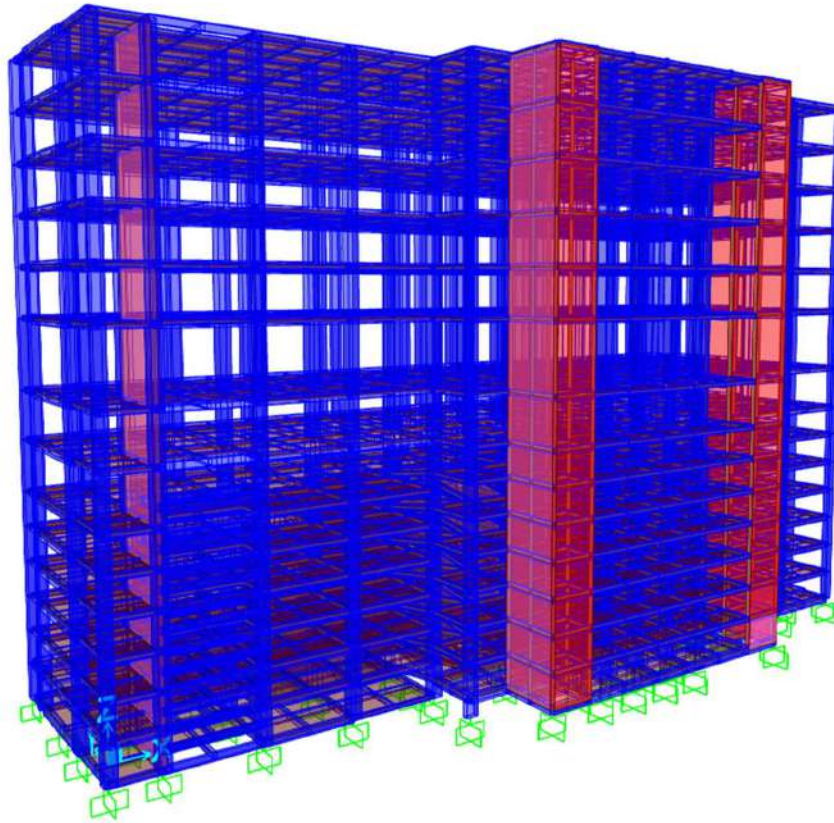
4.6.1 Planning Model 1

The building structure in Model 1 is a regular building structure and uses the Special Moment Frame System (SMFS) as its main system. The building structure has a total height of 53.5 meters and consists of 13 floors. The B to 6 floors is used as parking and have a height of 1.5 meters, while the other floors have a height of 4.2 meters.

In the middle of the building, there is a void that serves as a lift access and two stair accesses. The void used as lift access has dimensions of 7 meters in length and 4 meters in width. Meanwhile, the other two voids used as stair accesses have dimensions of 5 meters in length and 4 meters in width. The planning layout and elevation of model 1 can be seen in Figure 4. 6.



a)



b)

Figure 4. 6 Planning of Model 1. a) top view; b) 3D perspective.

4.6.1.1 Special Data for Model 1 Structure Planning

The planning data used in Model 1 exactly as stated in the previous subsection, the data includes:

1. The planned building structure is a regular building structure with 13 floors, functioning as a parking lot from the Basement to P7. Meanwhile, the other floors are designated as a hospital.
2. Building used as study namely the Kasih Ibu Hospital Building.
3. The building is located in the city of Surakarta, Central Java.
4. Risk Category IV.
5. Category Desain Seismic D (KDS D)

4.6.2 Planning Model 2

The building structure in Model 1 is a regular building structure and uses the Special Moment Frame System (SMFS) as its main system. The building structure has a total height of 53.5 meters and consists of 13 floors. The B to 6 floors are used as parking and have a height of 1.5 meters, while the other floors have a height of 4.2 meters.

The building structure planning in Model 2 is the same; the only difference is the change of axis from the strong axis to the weak axis on the main beams in the parking floor. Here is an example of the calculation of beam dimension estimates in the structure of model 2. The dimensions of the weak axis beams in Model 2 can be seen in Table 4.3.

Balok B1-A-Rotated

Length (L) = 8000mm

Height of the beam (H) = $\frac{1}{12} \times L$
= $\frac{1}{12} \times 8000 = 500 \text{ mm}$

H used = 500 mm

Table 4. 3 Weak axis beam dimension

Weak Axis Beam					
Beam	Type	L	Cross-sectional Dimensions		
			H min	H used	B
B1-A-Rotated	Main Beam	8000	500	500	800
B2-A-Rotated	Main Beam	8000	500	500	650
B2-c-Rotated	Main Beam	8000	500	500	650
B3-Rotated	Main Beam	8000	500	500	600
T1-Rotated	Main Beam	10000	625	650	800
T2-Rotated	Main Beam	8000	500	500	650
T3-Rotated	Main Beam	8000	500	500	600

4.6.2.1 The planning data used in Model 2 are as follows:

1. The building structure uses the Special Moment Resisting Frame System (SMRFS).
 2. The concrete quality used for beams, columns, and concrete floor slabs is $f_c' 30 \text{ MPa}$.
 3. The reinforcement steel material uses BJTD 42.
 4. The cross-section of the beam on the parking floor is rotated 90° , becoming a weak axis beam.
 5. The height of the weak axis beam is adjusted according to the SNI 2847: 2019 standard.
1. The first step is to collect data about the soil and topography of the site where the structure will be built. This data is then used to determine the earthquake load and soil type according to the location where the building stands, in this case study the city of Surakarta, Central Java.
 2. Once the data is collected, a 3D model of the structure is created using SAP 2000, a structural analysis software.
 3. The next step is to design the beams for the structure. In this research, we made changes to the beam axes into strong axis and weak axis, and did not make changes to the existing beam dimensions.
 4. After the beams were designed, compared the structural behavior of the two types of beams starting from irregularity, Story Drift and Drift Ratio, and P-Delta.

4.7 Structural Loading

The load planning on the structure is calculated based on the Minimum Design Load Guidelines and related criteria for buildings (SNI 1727:2020).

4.7.1 Dead load

Dead load is the weight of all the permanent construction materials of a building, such as columns, beams, walls, and also includes all additional finishing elements, machines, and fixed equipment that are an integral part of the building. The weight of the construction materials used in the planning can be seen in Table 4.4.

Table 4. 4 Weight of construction materials.

Dead Load	The size of load
Reinforced concrete	2400 kg/m ³
Rebar steel	7850 kg/m ³
Ceiling and hanger	18 kg/m ²
Mixture /cm of cement	21 kg/m ²
Half-brick couple	250 kg/m ²
Ceramic floor covering	24 kg/m ²
Waterproof	5 kg/m ²
<i>Mechanical and Electrical</i>	40 kg/m ²

The dead load in this planning consists of the self-weight of the structure and additional dead loads acting on the structure, which will be explained in the following item:

4.7.1.1. Shelf Weight

Self-weight is the weight of the structural components, excluding any additional elements that do not bear any load. The self-weight calculation has been automatically computed by the structural analysis program.

4.7.1.2. Additional dead load for the floor slab

On the floor slab, there is an additional dead load that serves as the finishing of the floor slab. This load consists of floor tiles, cement mortar from the grout between the slab and the tiles, ceiling along with its hangers, Mechanical and Engineering, and others.

4.7.1.3. Wall Weight

In planning, walls are not considered components of the structure (except for structural walls) because they do not bear loads, so walls are made as additional loads. The wall load is treated as a line load supported by beams connected to each floor. In this planning, the wall load is only placed on the outermost beams, because the inner walls use partitions.

4.7.2. Live Load

Live load is the load that occurs due to the occupancy of a building or other structure that does not include construction loads and dead loads. All live loads have the characteristic of being able to shift or move. Generally, this load acts in a vertical downward direction, but it can also act horizontally at times. The planning of live loads refers to SNI 1727:2013, which can be seen in Table 4.5.

Table 4. 5 Structure Live load

Live Load	Wight (kg/m ²)
Office floors, hotels, apartments, schools, dormitories, market, hospital.	250
The load of life on stairs and landings.	300
Minimum live load on the roof.	100
The load of life in the meeting room.	400

4.7.3. Earthquake load

Eearthquake load is the load that arises due to the movement of the ground where the structure stands. The loading of the structure due to earthquake loads is based on SNI 1726:2019. The earthquake load analysis consists of three analyses, namely the Equivalent Lateral Force analysis, the Response Spectrum Modal analysis, and the Time History Response procedure. The earthquake load planning in this final project uses the Modal Response Spectrum analysis and is checked with the Equivalent Lateral Force analysis.

4.8 Earthquake Design Response Spectrum

Based on SNI 1726:2019, the earthquake design response spectrum must be created first. With the data of bedrock acceleration $SS = 0.814$, $S1 = 0.395$, and $PGA = 0.515$ on special soil conditions located in Solo, Central Java. The values are:

- F_a = 1,248 (Table 3.1)
- F_v = 2,4 (Table 3.2)
- S_{MS} = 1,016 (Equation 3-14)
- S_{MI} = 1,44 (Equation 3-15)
- S_{DS} = 0,677 (Equation 3-16)
- S_{DI} = 0,637 (Equation 3-20)
- T_0 = 0,187 (Equation 3-21)
- T_s = 0,9 (Equation 3-18 and Equation 3-19)
- T_L = 20 (Figure 3.4)

The spectrum response is plotted into a graph as shown in Figure 4.7.

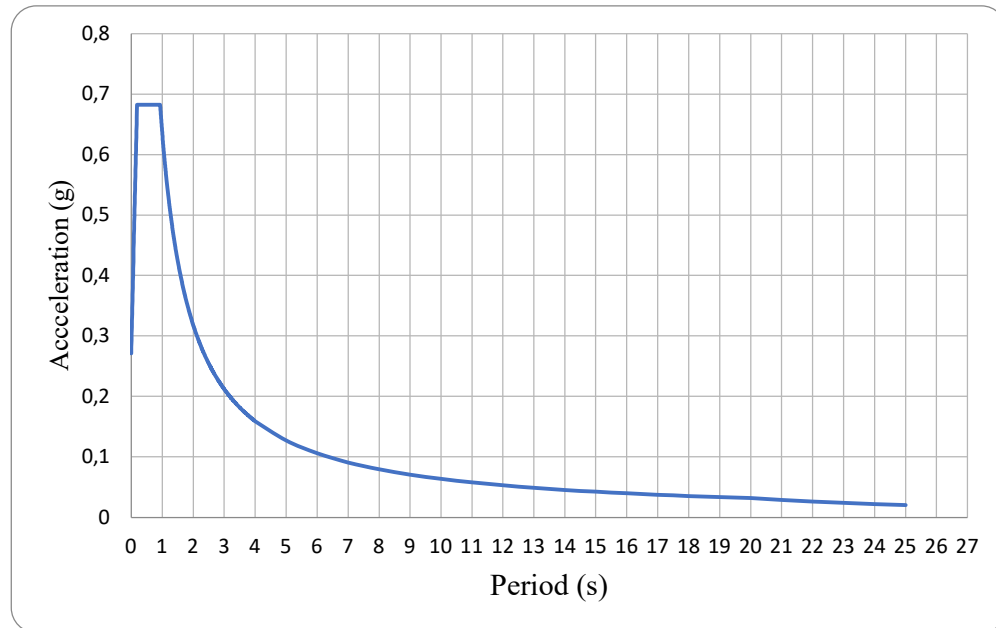


Figure 4. 7 Design Spectral Response Graph.

4.9. Load Combination

All dead loads, live loads, and earthquake loads are calculated with magnification factors and load combinations that are input into the structural analysis program based on SNI 1726:2019. For this modelling, a value of $\rho = 1.3$ was used, obtained from seismic design D, and a value of $\text{SDS} = 0.839$. The value of $\rho = 1.3$ was chosen because if $\rho = 1$ is used, there are still floor shear forces on several levels that do not meet the requirement that the floor shear force (V) must be greater than 35% of the base shear. Therefore, the value of $\rho = 1.3$ was chosen, resulting in the load combination as shown in Table 4.6.

Table 4. 6 Load Combination.

Combines	Coefficient		Coefficient		Coefficient		Coefficient	
Combines 1	1.4	DL	0	LL	0	EX	0	EY
Combines 2	1.2	DL	1.6	LL	0	EX	0	EY
Combines 3	1.42	DL	1	LL	0.39	EX	1.3	EY
Combines 4	0.98	DL	1	LL	-0.39	EX	-1.3	EY
Combines 5	1.08	DL	1	LL	0.39	EX	-1.3	EY
Combines 6	1.32	DL	1	LL	-0.39	EX	1.3	EY
Combines 7	1.42	DL	1	LL	1.3	EX	0.39	EY
Combines 8	0.98	DL	1	LL	-1.3	EX	-0.39	EY
Combines 9	1.32	DL	1	LL	1.3	EX	-0.39	EY
Combines 10	1.08	DL	1	LL	-1.3	EX	0.39	EY
Combines 11	1.12	DL	0	LL	0.39	EX	1.3	EY
Combines 12	0.68	DL	0	LL	-0.39	EX	-1.3	EY
Combines 13	0.78	DL	0	LL	0.39	EX	-1.3	EY
Combines 14	1.02	DL	0	LL	-0.39	EX	1.3	EY
Combines 15	1.12	DL	0	LL	1.3	EX	0.39	EY
Combines 16	0.68	DL	0	LL	-1.3	EX	-0.39	EY
Combines 17	1.02	DL	0	LL	1.3	EX	-0.39	EY
Combines 18	0.78	DL	0	LL	-1.3	EX	0.39	EY

CHAPTER V

RESULTS AND DISCUSSION

5.1 Overview

In this chapter, the results of the comparative analysis between strong axis beams and weak axis beams will be discussed. Structural modelling using a SRPMK system is differentiated by rotating the axis of all beams in the parking building, which varies in each of its models. All forms of loading input, earthquake zone classification, and the number of levels is the same.

5.2 Analysis Results

5.2.1 Results of Model 1 Structure Analysis

In Model 1, the combination of the variance of mass participation has reached 100 percent (*Sum Ux* and *Sum Uy*) at mode 18, thus mass participation has met the requirements. Data on mass participation can be seen in Table 5.1.

Table 5. 1 Data output period of Model 1 structural analysis program.

Mode	Period (sec)	Sum UX	Sum UY	Sum RZ
1	1,719	0,7846	0	0.0002
2	1,398	0,7846	0,7847	0.0002
3	1,140	0,7847	0,7847	0.7811
4	0,428	0,8908	0,7847	0.7811
5	0,415	0,8908	0,8949	0.7811
6	0,275	0,8908	0,8949	0.889
7	0,235	0,9335	0,8949	0.889
8	0,208	0,9335	0,9369	0.889
9	0,190	0,9335	0,9369	0.9317
10	0,150	0,9598	0,9369	0.9317
11	0,136	0,9598	0,9621	0.9317
12	0,124	0,9598	0,9621	0.9574
13	0,086	0,97032	0,97946	0.9574
14	0,082	0,97708	0,99122	0,97449

Table 5.1 continued Data output period of Model 1 structural analysis program.

Mode	Period (sec)	Sum UX	Sum UY	Sum RZ
15	0,082	0,98384	1	0,97767
16	0,081	0,99060	1	0,98519
17	0,081	0,99737	1	0,98519
18	0,077	1	1	0,99174
19	0,077	1	1	0,99175

5.2.1.1 Story Shear

Story shear in the context of seismic analysis refers to the lateral force acting on each level of a structure due to seismic loads. When analyzed using the Static Equivalent approach as defined by SNI 1726:2019, this means that the seismic effects are approximated through an equivalent static lateral force that is calculated and distributed across each story of the building. Example of calculating the story shear at the 10th floor as follows:

$$C_{vx} = \frac{W_x h_x^k}{\sum_{i=1}^n W_i h_i^k}$$

Where :

- C_{vx} = Faktor distribusi vertikal
- V_x = gaya lateral desain total
- W_i or W_x = Wight of floor
- h_i or h_x = Height of loor
- K = The exponent associated with the structural period

is defined as follows :

For structures with $T \leq 0,5$ seconds, $k = 1$

For structures with $T \geq 2,5$ seconds, $k = 2$

For structures with $0,5 < T < 2,5$ seconds, $k = 2$ or determined through linear interpolation between 1 and 2.

10th floor data :

- T = 1,718 s
- K = 1,609 from linear interpolation between

$$\begin{aligned}
H_{10} &= 40,9 \text{ m} \\
W_{10} &= 16626,247 \text{ kN} \\
W_{10}h_{10}^k &= 16626,247 \times 40,9^{1,609} \\
&= 6518349 \text{ kNm} \\
\sum_{i=1}^n W_i h_i^k &= 52580975 \text{ kNm} \\
C_{V10} &= \frac{6518349 \text{ kNm}}{52580975 \text{ kNm}} \\
&= 0,124 \\
V_{10} &= 0,124 + 0,468 \\
&= 0,592
\end{aligned}$$

The control of Story shear for Model 1 is presented in Tables 5.2.

Table 5. 2 Story shear for Model 1

Storey	Weight of floor (KN)	H (m)	K	wi.hi ^k	Cvx	Vx	Vi/Vi+1
Rooftop	13505,099	53,5	1,60905	8156530	0,155	0,155	
12	16626,247	49,3	1,60905	8803788	0,167	0,323	2,079
11	16626,247	45,1	1,60905	7628638	0,145	0,468	1,450
10	16626,247	40,9	1,60905	6518349	0,124	0,592	1,265
9	16626,247	36,7	1,60905	5475454	0,104	0,696	1,176
8	16626,247	32,5	1,60905	4502878	0,086	0,781	1,123
7	19304,95	26,5	1,60905	3764805	0,072	0,853	1,092
6	16581,713	22,3	1,60905	2449736	0,047	0,900	1,055
P7	5062,594	19,4	1,60905	597737,5	0,011	0,911	1,013
P6M	10651,995	17,9	1,60905	1104928	0,021	0,932	1,023
P6	6027,517	16,4	1,60905	543103,9	0,010	0,942	1,011
P5M	10161,6065	14,9	1,60905	784653,2	0,015	0,957	1,016
P5	6050,137	13,4	1,60905	393852,1	0,0075	0,965	1,008
P4M	12354,0515	11,9	1,60905	664382,6	0,013	0,977	1,013
P4	6027,517	10,4	1,60905	260973,2	0,0050	0,982	1,005
P3M	11473,2865	8,9	1,60905	386637,4	0,0074	0,990	1,007
P3	7733,648	7,4	1,60905	193651,9	0,0037	0,993	1,004
P2M	11572,084	5,9	1,60905	201257	0,0038	0,997	1,004
P2	6338,249	4,4	1,60905	68756,99	0,0013	0,998	1,001
P1M	10832,672	3	1,60905	63452,33	0,0012	0,9997	1,001
P1	9067,4965	1,5	1,60905	17411,13	0,00033	1,0	1,000
B	9534,489	0	1,60905	0	0	1	1,000
Total	255410,34			52580975	1		

5.2.1.2 Soft Story Stiffness Irregularity

Based on SNI 1726:2019, a soft story is defined as a level where its lateral stiffness is less than 70 percent of the lateral stiffness of the level above it or less than 80 percent of the average lateral stiffness of the three levels above it. For this purpose, it is necessary to examine the building's structure to determine whether it has a soft-story irregularity or not. Example of calculating the structural stiffness at the 10th floor due to an earthquake load in the x-direction is as follows:

Structural stiffness conditions:

$$K_i > 0,7 K_{i+1} \rightarrow \text{Regular}$$

$$K_i > 0,8 \text{ Avg}(K_{i+1} + K_{i+2} + K_{i+3}) \rightarrow \text{Regular}$$

Where :

$$\Delta i = U_{10} - U_9 = 0,076554 - 0,07245 = 0,004104 \text{ m}$$

$$K_i = \frac{V_x}{\Delta i} = \frac{4}{0,004104} = 974,658 \text{ Kn/m}$$

$$K_{i+1} = k_{i11} = 726,216 \text{ Kn/m}$$

$$974,658 > 0,7 \times 726,216$$

$$974,658 > 405,679 \rightarrow \text{Regular}$$

Explanation:

Z = Height of floor

F_x = Vertical load on and level x

V_x = Total vertical load at level x

Δi = Drift at level x

The control of soft story irregularity in the x and y directions for Model 1 is presented in Tables 5.

Table 5. 3 Soft story stiffness irregularity control in x-direction earthquake

Storey	Z	FX	Accumulative	U1	Δi	K	Cek	Conclusion
	(mm)	(kN)	Vx (kN)	mm	(m)	(Kn/mm)	70%Ki+1	
Rooftop	52000	1	1	0,086864	0,002728	366,5689	0	Regular
12	47800	1	2	0,084136	0,003451	579,5422	256,5982	Regular
11	43600	1	3	0,080685	0,004131	726,2164	405,6795	Regular
10	39400	1	4	0,076554	0,004104	974,6589	508,3515	Regular
9	35200	1	5	0,07245	0,007207	693,7699	682,2612	Regular
8	31000	1	6	0,065243	0,009312	644,3299	485,639	Regular
7	25000	1	7	0,055931	0,008406	832,7385	451,0309	Regular
6	20800	1	8	0,047525	0,006384	1253,133	582,917	Regular
P7	17900	1	9	0,041141	0,003551	2534,497	877,193	Regular
P6M	16400	1	10	0,03759	0,00361	2770,083	1774,148	Regular
P6	14900	1	11	0,03398	0,003678	2990,756	1939,058	Regular
P5M	13400	1	12	0,030302	0,00378	3174,603	2093,529	Regular
P5	11900	1	13	0,026522	0,0038	3421,053	2222,222	Regular
P4M	10400	1	14	0,022722	0,003841	3644,884	2394,737	Regular
P4	8900	1	15	0,018881	0,003839	3907,268	2551,419	Regular
P3M	7400	1	16	0,015042	0,003771	4242,906	2735,087	Regular

Continued Table 5.3 Soft story stiffness irregularity control in x-direction earthquake.

Storey	Z	FX	Accumulative	U1	Δi	K	Cek	Conclusion	cek	Conclusion
	(mm)	(kN)	Vx (kN)	mm	(m)	(Kn/mm)	70%Ki+1		80%(avgKi+1)	
P3	5900	1	17	0,011271	0,00357	4761,905	2970,034	Regular	3145,349	Regular
P2M	4400	1	18	0,007701	0,003158	5699,81	3333,333	Regular	3443,221	Regular
P2	2900	1	19	0,004543	0,002227	8531,657	3989,867	Regular	3921,232	Regular
P1M	1500	1	20	0,002316	0,001589	12586,53	5972,16	Regular	5064,899	Regular
P1	0	1	21	0,000727	0,000727	28885,83	8810,573	Regular	7151,466	Regular
B	-1500	0	0	0	0		0			

Table 5. 4 Soft story stiffness irregularity control in y-direction earthquake.

Storey	Z	FX	Accumulative	U2	Δi	K	Cek	Conclusion	cek	Conclusion
	(mm)	(kN)	Vx (kN)	m	(m)	(Kn/m)	70%Ki+1		80%(avgKi+1)	
Rooftop	52000	1	1	0,055643	0,002811	355,7453	0	Regular	0	Regular
12	47800	1	2	0,052832	0,003219	621,311	249,0217	Regular	284,5962	Regular
11	43600	1	3	0,049613	0,002933	1022,844	434,9177	Regular	390,8225	Regular
10	39400	1	4	0,04668	0,001133	3530,45	715,9905	Regular	533,3066	Regular
9	35200	1	5	0,045547	0,001601	3123,048	2471,315	Regular	1379,895	Regular
8	31000	1	6	0,043946	0,001225	4897,959	2186,134	Regular	2047,024	Regular
7	25000	1	7	0,042721	0,001519	4608,295	3428,571	Regular	3080,389	Regular

Continued Table 5.4 Soft story stiffness irregularity control in Y-direction earthquake.

Storey	Z	FX	Accumulative	U2	Δi	K	Cek	Conclusion	cek	Conclusion
	(mm)	(kN)	Vx (kN)	m	(m)	(Kn/m)	70%Ki+1		80%(avgKi+1)	
6	20800	1	8	0,041202	0,003637	2199,615	3225,806	Irregular	3367,814	Irregular
P7	17900	1	9	0,037565	0,004863	1850,709	1539,731	Regular	3121,565	Irregular
P6M	16400	1	10	0,032702	0,013139	761,0929	1295,497	Irregular	2308,965	Irregular
P6	14900	1	11	0,019563	0,002835	3880,071	532,7651	Regular	1283,045	Regular
P5M	13400	1	12	0,016728	0,002596	4622,496	2716,049	Regular	1731,166	Regular
P5	11900	1	13	0,014132	0,002394	5430,242	3235,747	Regular	2470,309	Regular
P4M	10400	1	14	0,011738	0,002173	6442,706	3801,17	Regular	3715,416	Regular
P4	8900	1	15	0,009565	0,002016	7440,476	4509,894	Regular	4398,785	Regular
P3M	7400	1	16	0,007549	0,001863	8588,298	5208,333	Regular	5150,247	Regular
P3	5900	1	17	0,005686	0,001708	9953,162	6011,809	Regular	5992,395	Regular
P2M	4400	1	18	0,003978	0,001512	11904,76	6967,213	Regular	6928,516	Regular
P2	2900	1	19	0,002466	0,001151	16507,38	8333,333	Regular	8118,993	Regular
P1M	1500	1	20	0,001315	0,000906	22075,06	11555,17	Regular	10230,75	Regular
P1	0	1	21	0,000409	0,000409	51344,74	15452,54	Regular	13463,25	Regular
B	-1500	0	0	0	0	0				

From Tables 5.3 and 5.4, there is irregularity in the lateral stiffness percentage of each floor, which is less than 70% for the floor above or less than 80% for the average of the three floors above on floor P6 and floor 6 in the y direction. The building structure in Model 1 exhibits soft-story stiffness irregularity in both the x and y directions. Based on SNI 1726:2019, if there is no drift ratio up to 1.3 times compared to the drift ratio of the floor above, then vertical irregularities Type 1a (soft story stiffness irregularity) and Type 1b (excessive soft story irregularity) can be ignored.

5.2.1.3 Mass Irregularity

Based on SNI 1726:2012, it is stated that a mass irregularity is defined to exist if there is a level that has an effective mass greater than 150 percent of the effective mass of the nearby level. Therefore, an inspection of the building structure is necessary to determine whether it has mass irregularity or not. Example of mass irregularity calculation on the 10th floor is as follows:

Condition of Mass Irregularity:

$W_i > 150\% W_{i+1} \rightarrow \text{Irregular}$

$W_{i10} > 150\% W_{i11} \rightarrow 16626,247 < 24939,3705 \rightarrow \text{Regular}$

The control of mass irregularity for Model 1 can be seen in Table 5.5.

Table 5. 5 Mass irregularity control for Model 1

Storey	Weight of floor (KN)	150%W _{i+1}	Conclusion
Rooftop	13505,099	0	
12	16626,247	20257,6485	Regular
11	16626,247	24939,3705	Regular
10	16626,247	24939,3705	Regular
9	16626,247	24939,3705	Regular
8	16626,247	24939,3705	Regular
7	19304,95	24939,3705	Regular
6	16581,713	28957,425	Regular
P7	5062,594	24872,5695	Regular
P6M	10651,995	7593,891	Irregular
P6	6027,517	15977,9925	Regular
P5M	10161,6065	9041,2755	Irregular

Continued Table 5.5 Mass irregularity control for Model 1

Storey	Weight of floor (KN)	150%W _{i+1}	Conclusion
P5	6050,137	15242,40975	Regular
P4M	12354,0515	9075,2055	Irregular
P4	6027,517	18531,07725	Regular
P3M	11473,2865	9041,2755	Irregular
P3	7733,648	17209,92975	Regular
P2M	11572,084	11600,472	Regular
P2	6338,249	17358,126	Regular
P1M	15059,99	9507,3735	Irregular
P1	9067,4965	22589,985	Regular
B	9534,489	13601,24475	Regular
Total	259637,658		

From Table 5.5, there is an effective mass percentage exceeding 150% of the level above it, specifically on floors P1M, P3M, P4M, P5M, and P6M, indicating that the building structure in Model 1 does not have mass irregularity. Based on SNI 1726:2019, if there is no drift ratio up to 1.3 times compared to the drift ratio of the floor above, then vertical irregularities Type 2 b (mass irregularity) can be ignored.

5.2.1.4 Geometric Irregularities

Based on SNI 1726:2019, vertical geometric irregularity in building structures is defined as a condition where the horizontal dimensions of the seismic force-resisting system at each level exceed 130 percent. Example of calculating geometric irregularity on the 10th floor is as follows:

$$D_i < 130\% D_{i+1} \rightarrow \text{Regular}$$

$$D_{i10} < 130\% D_{i11}$$

$$43,6 \text{ m} < 56,68 \text{ m} \rightarrow \text{Regular}$$

Explanation:

D_i = The width of the earthquake-resistant portal from the first column to the farthest column end.

The control of geometric irregularity in the x and y directions for Model 1 is presented in Tables 5.6 and 5.7.

Table 5. 6 Geometric Irregularities control in x-direction earthquake

Storey	Di (m)	130%di+1	Conclusion
Rooftop	43,6	0	
12	43,6	56,68	Regular
11	43,6	56,68	Regular
10	43,6	56,68	Regular
9	43,6	56,68	Regular
8	43,6	56,68	Regular
7	43,6	56,68	Regular
6	43,6	56,68	Regular
P7	43,6	56,68	Regular
P6M	43,6	56,68	Regular
P6	43,6	56,68	Regular
P5M	43,6	56,68	Regular
P5	43,6	56,68	Regular
P4M	43,6	56,68	Regular
P4	43,6	56,68	Regular
P3M	43,6	56,68	Regular
P3	43,6	56,68	Regular
P2M	43,6	56,68	Regular
P2	43,6	56,68	Regular
P1M	43,6	56,68	Regular
P1	43,6	56,68	Regular
B	43,6	56,68	Regular

Table 5. 7 Geometric Irregularities control in y-direction earthquake

Storey	Di (m)	di/di+1	Conclusion
Rooftop	24,792	0	
12	24,792	32,2296	Regular
11	24,792	32,2296	Regular
10	24,792	32,2296	Regular
9	24,792	32,2296	Regular
8	24,792	32,2296	Regular
7	24,792	32,2296	Regular
6	24,792	32,2296	Regular
P7	24,792	32,2296	Regular
P6M	24,792	32,2296	Regular

Continued Table 5.7 Geometric Irregularities control in x-direction earthquake

Storey	Di (m)	di/di+1	Conclusion
P6	24,792	32,2296	Regular
P5M	24,792	32,2296	Regular
P5	24,792	32,2296	Regular
P4M	24,792	32,2296	Regular
P4	24,792	32,2296	Regular
P3M	24,792	32,2296	Regular
P3	24,792	32,2296	Regular
P2M	24,792	32,2296	Regular
P2	24,792	32,2296	Regular
P1M	24,792	32,2296	Regular
P1	24,792	32,2296	Regular
B	24,792	32,2296	Regular

From Tables 5.6 and 5.7, there is no vertical geometric irregularity on each floor of the building structure in Model 1, defined as a condition where the horizontal dimensions of the earthquake resistance system on each level exceed 130 percent.

5.2.1.5 Story Drift and Drift Ratio

The inter-floor displacement is the difference in the deflection value of the building between floor i and the floor below it. Based on the SNI 1726:2019 regulations, the inter-story drift control only has one performance limit, which is the ultimate limit state. The design-level inter-story drift must not exceed the permissible inter-story drift multiplied by the redundancy factor value. Story drift and drift ratio for the 10th floor can be calculated using the following equation:

$$\frac{\delta_{Max}}{\delta_{Average}} < 1,2 \rightarrow \text{Regular}$$

$$\begin{aligned} \delta A_{10} &= \text{Def. Joint } Ax_{10} - \text{Def. Joint } Ax_9 \\ &= 0,10428 - 0,090621 \\ &= 0,013659 \end{aligned}$$

$$\delta B_{10} = \text{Def. Joint } Bx_{10} - \text{Def. Joint } Bx_9$$

$$= 0,098323 - 0,085387$$

$$= 0,012936$$

$$\delta_{Max} = 0,013659$$

$$\delta_{Average} = 0,013297$$

$$\frac{\delta_{Max}}{\delta_{Average}} = \frac{0,013659}{0,013297}$$

$$= 1,0271$$

$$= 1,027 < 1,2 \rightarrow \text{Regular}$$

Where:

δ = Story drift

Tables 5.8 and 5.9 present the results of the deviation values between floors for Model.

Table 5. 8 Story Drift and Drift Ratio X – direction

Storey	Def. Joint Ax	δA	Def. Joint Bx	δB	δ Max	δ Average	Rasio δ Max / δ Average	Conclusion	Ax	Ax used	5% Ax
Rooftop	0,153	0,007	0,1534	0,0078	0,0078	0,00781	1,003	Regular	0,699	1	0,05
12	0,145	0,0097	0,1455	0,0097	0,0097	0,00972	1,002	Regular	0,698	1	0,05
11	0,135	0,011	0,1358	0,0112	0,0112	0,01125	1,000	Regular	0,694	1	0,05
10	0,124	0,0124	0,1245	0,0124	0,0124	0,01245	1,000	Regular	0,695	1	0,05
9	0,112	0,013	0,1121	0,0135	0,0136	0,01360	1,001	Regular	0,695	1	0,05
8	0,098	0,021	0,0985	0,0219	0,0219	0,02192	1,000	Regular	0,695	1	0,05
7	0,076	0,0150	0,0765	0,0149	0,0150	0,01498	1,002	Regular	0,697	1	0,05
6	0,061	0,010	0,061	0,0102	0,0103	0,01029	1,002	Regular	0,697	1	0,05
P7	0,051	0,0051	0,0513	0,0052	0,0052	0,00520	1,001	Regular	0,696	1	0,05
P6M	0,046	0,0051	0,0461	0,0051	0,0051	0,00514	1,001	Regular	0,696	1	0,05
P6	0,040	0,0050	0,0409	0,0050	0,0050	0,00508	1,000	Regular	0,695	1	0,05
P5M	0,035	0,0050	0,03590	0,0050	0,0050	0,00506	1,000	Regular	0,694	1	0,05
P5	0,030	0,004	0,0308	0,0049	0,0049	0,00498	1,001	Regular	0,696	1	0,05
P4M	0,025	0,0048	0,02587	0,0048	0,0048	0,00483	1,002	Regular	0,697	1	0,05
P4	0,021	0,0046	0,0210	0,0046	0,0046	0,00465	1,000	Regular	0,695	1	0,05
P3M	0,016	0,0043	0,0163	0,0043	0,0043	0,00437	1,000	Regular	0,695	1	0,05
P3	0,011	0,0039	0,0119	0,0039	0,0039	0,00393	1,000	Regular	0,694	1	0,05
P2M	0,008	0,0033	0,0080	0,0033	0,0033	0,00332	1,002	Regular	0,697	1	0,05
P2	0,004	0,0023	0,0047	0,0028	0,0028	0,00261	1,095	Regular	0,833	1	0,05

Continued Table 5.8 Story Drift and Drift Ratio X – direction

Storey	Def. Joint Ax	δA	Def. Joint Bx	δB	δ Max	δ Average	Rasio δ Max / δ Average	Conclusion	Ax	Ax used	5% Ax
P1M	0,0023	0,0017	0,0018	0,0012	0,0017	0,00149	1,139	Regular	0,901	1	0,05
P1	0,0005	0,00065	0,00055	0,0005	0,0006	0,00060	1,078	Regular	0,807	1	0,05
B	0	0	0	0	0	0					

Table 5. 9 Story Drift and Drift Ratio Y – direction

Storey	Def. Joint Ay	δA	Def. Joint By	δB	δ Max	δ Average	Rasio δ Max / δ Average	Conclusion	Ax	Ax used	5% Ax	Ey
Rooftop	0,1403	0,0104	0,1325	0,0099	0,0104	0,010215	1,023494	Regular	0,727	1	0,0500	0,916
12	0,1298	0,0123	0,1226	0,0116	0,0123	0,01200	1,0259913	Regular	0,731	1	0,05000	0,921
11	0,1175	0,0132	0,1109	0,0126	0,0132	0,01295	1,026601	Regular	0,731	1	0,05000	0,922
10	0,10428	0,0136	0,0983	0,0129	0,0136	0,01329	1,0271855	Regular	0,7327	1	0,05000	0,923
9	0,0906	0,0137	0,0853	0,0130	0,0137	0,01340	1,0280917	Regular	0,7340	1	0,05000	0,924
8	0,0768	0,0206	0,0723	0,0194	0,0206	0,02006	1,0285101	Regular	0,734	1	0,0500	0,925
7	0,0562	0,0135	0,0528	0,0126	0,0135	0,01307	1,033242	Regular	0,741	1	0,05000	0,934
6	0,0427	0,0091	0,0402	0,0080	0,0091	0,00860	1,0654270	Regular	0,788	1	0,05000	0,993
P7	0,0335	0,0035	0,0321	0,0038	0,0038	0,00369	1,032327	Regular	0,740	1	0,05000	0,932

Continued Table 5.9 Story Drift and Drift Ratio Y – direction

Storey	Def. Joint Ay	δA	Def. Joint By	δB	δ Max	δ Average	Rasio δ_{Max} / δ Average	Conclusion	Ax	Ax used	5% Ax	Ey
P6M	0,0299	0,0037	0,0283	0,0034	0,0037	0,00360	1,050561	Regular	0,766	1	0,05000	0,965
P6	0,0261	0,0033	0,0249	0,0034	0,0034	0,00335	1,014609	Regular	0,714	1	0,05000	0,900
P5M	0,0228	0,0035	0,0215	0,0030	0,0035	0,0033	1,062121	Regular	0,7834	1	0,05000	0,987
P5	0,0193	0,0030	0,0184	0,0031	0,0031	0,00308	1,013942	Regular	0,713	1	0,05000	0,899
P4M	0,0163	0,0031	0,0153	0,0027	0,0031	0,00294	1,067503	Regular	0,791	1	0,05000	0,997
P4	0,0131	0,0026	0,0125	0,0027	0,0027	0,0027	1,004063	Regular	0,700	1	0,05000	0,882
P3M	0,0104	0,0027	0,0098	0,0024	0,0027	0,00259	1,057324	Regular	0,776	1	0,05000	0,978
P3	0,0077	0,0022	0,0074	0,0023	0,0023	0,00227	1,030728	Regular	0,7377	1	0,05000	0,929
P2M	0,0055	0,0020	0,0050	0,002	0,0020	0,00203	1,015263	Regular	0,715	1	0,050000	0,901
P2	0,0034	0,0017	0,0030	0,0015	0,0017	0,00167	1,051595	Regular	0,7679	1	0,05000	0,967
P1M	0,0016	0,0012	0,0014	0,0010	0,0012	0,00117	1,087362	Regular	0,8210	1	0,05000	1,034
P1	0,0004	0,00041	0,0004	0,0004	0,0004	0,00040	1,0195599	Regular	0,7218	1	0,05000	0,909
B	0	0	0	0	0	0						

Based on the results obtained from Table 5.8 and Table 5.9, it can be observed that there is no torsional irregularity between the floors of the building structure in the model. This is indicated by the ratio of the maximum drift to the average drift not exceeding the specified limit of 1.2, as stipulated in SNI 1726:2019 regarding the provisions for detecting the presence of torsional irregularity. The absence of torsional irregularity indicates a uniform distribution of structural stiffness, allowing the structure to exhibit more stable behaviour when subjected to seismic loads. In accordance with the provisions of SNI 1726:2019, if the ratio between the maximum displacement and the average displacement on a floor does not exceed 1.2, then the structure can be categorized as a structure without torsional irregularity, which is important to ensure the building's performance remains optimal in facing lateral loads due to an earthquake.

5.2.1.6 Internal Angle Irregularity

Based on SNI 1726:2019, The internal angular irregularity is defined to be present if both projections of the internal angle structure plan are greater than 15 percent of the dimension of the internal structure plan in the specified direction can be seen on figure 5.1.

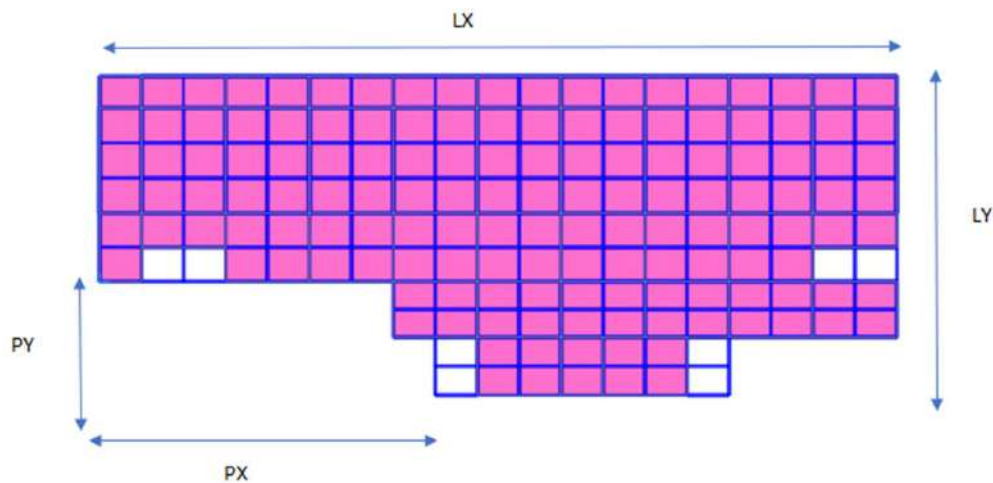


Figure 5. 1 Internal angular irregularity

Example of calculating internal angular irregularity on the 10th floor is as follows:

Condition of Mass Irregularity:

$P_x > 15\% L_x \rightarrow \text{Irregular}$

$32000 > 15\% 76000$

$32000 > 11400 \rightarrow \text{Irregular}$

$P_y > 15\% L_y \rightarrow \text{Irregular}$

$30300 > 15\% 10800$

$30300 > 4545 \rightarrow \text{Irregular}$

The control of internal angular irregularity for Model 1 is presented in Tables 5.10.

Table 5. 10 Internal angle irregularity

Storey	LX (mm)	PX (mm)	15%LX	STATUS	LY (mm)	PY (mm)	15%LY	STATUS
Rooftop	76000	32000	11400	Irregular	30300	10800	4545	Irregular
12	76000	32000	11400	Irregular	30300	10800	4545	Irregular
11	76000	32000	11400	Irregular	30300	10800	4545	Irregular
10	76000	32000	11400	Irregular	30300	10800	4545	Irregular
9	76000	32000	11400	Irregular	30300	10800	4545	Irregular
8	76000	32000	11400	Irregular	30300	10800	4545	Irregular
7	76000	32000	11400	Irregular	30300	10800	4545	Irregular
6	76000	32000	11400	Irregular	30300	10800	4545	Irregular
P7	76000	32000	11400	Irregular	30300	10800	4545	Irregular
P6M	76000	36000	11400	Irregular	14400	12000	2160	Irregular
P6	48000	16000	7200	Irregular	14400	12000	2160	Irregular
P5M	76000	36000	11400	Irregular	19500	1000	2925	Irregular
P5	48000	16000	7200	Irregular	14400	12000	2160	Irregular
P4M	76000	32000	11400	Irregular	37500	18000	5625	Irregular
P4	48000	16000	7200	Irregular	14400	12000	2160	Irregular
P3M	76000	32000	11400	Irregular	37500	18000	5625	Irregular

Continued Table 5.10 Internal angle irregularity

Storey	LX (mm)	PX (mm)	15%LX	STATUS	LY (mm)	PY (mm)	15%LY	STATUS
P3	48000	16000	7200	Irregular	14400	12000	2160	Irregular
P2M	76000	32000	11400	Irregular	37500	18000	5625	Irregular
P2	48000	16000	7200	Irregular	14400	12000	2160	Irregular
P1M	76000	32000	11400	Irregular	37500	18000	5625	Irregular
P1	48000	16000	7200	Irregular	14400	12000	2160	Irregular
B	76000	32000	11400	Irregular	37500	18000	5625	Irregular

Based on the results presented in Table 5.10, it is known that there is a vertical irregularity in all floors of the building structure in Model 1. Referring to the provisions set forth in SNI 1726:2019, if there is a type 2 horizontal irregularity, characterized by a significant change in the distribution of stiffness or the horizontal geometry of the structure between floors, then structural elements such as diaphragms, chord elements, collectors, and their connections are required to be designed with an additional multiplier factor of 1.25. The increase in this factor aims to ensure that the capacity of these elements to transfer lateral forces due to an earthquake can be enhanced, thereby compensating for the horizontal irregularity that occurs and preventing potential excessive stress concentration on these elements. This provision is an effort to maintain the optimal performance of the structure in facing lateral loads that occur during an earthquake event, as well as to minimize the risk of partial collapse due to the weakness of diaphragm elements and their components. Thus, the application of this multiplier factor is in accordance with the earthquake resilience planning principles outlined in SNI 1726:2019 to enhance the overall reliability and safety of the building structure.

5.2.1.7 Influence of P-Delta Effect

Based on SNI 1726:2019, the P-delta effect must be considered for building structures that bear earthquake loads. However, the P-delta effect can be disregarded if the stability ratio is less than 0.1. The P-delta effect at the reviewed level must be calculated using the following equation:

$$\theta = \frac{P_x \Delta i_e}{V_x h_{sx} C_d}$$

$$\begin{aligned}\theta_{10} &= \frac{P_{x10} \Delta_{10} i_e}{V_{x1} h_{sx10} C_d} \\ &= \frac{1626,274 \times 12,46 \times 1,5}{2841,599 \times 4200 \times 5,5} = 0,005 < 0,1 \rightarrow \text{ok}\end{aligned}$$

Explanation :

C_d = deflation amplification factor

Δ_x = drift at level x (mm)

I_e = earthquake priority factor

P_x = Total vertical design load at and above level x (kN)

V_x = high level below level x (mm)

The control of the P-delta effect for the X and Y directions can be seen in Tables 5.11 and 5.12.

Table 5. 11 P-delta control in the x direction for Model 1

X Direction							Storey Drift Check			P-Delta Stability Check				
Storey	Px	Vx	Cd	Ie	hsx	U1		Δ	Δ Max	Δ < Δ Max	θ	θ max	θ < θ max	
	F3	F1			mm	m		mm	mm	mm				mm
	kN	kN												
Rooftop	13505,099	3480,120	5,5	1,5	4200	0,153376	153,4	7,79	32,31	OK	0,002	0,0909	OK	
12	16626,247	3837,909			4200	0,145583	145,6	9,76	32,31	OK	0,003	0,0909	OK	
11	16626,247	3325,616			4200	0,135827	135,8	11,25	32,31	OK	0,004	0,0909	OK	
10	16626,247	2841,599			4200	0,124577	124,6	12,46	32,31	OK	0,005	0,0909	OK	
9	16626,247	2386,961			4200	0,112115	112,1	13,59	32,31	OK	0,006	0,0909	OK	
8	16626,247	2024,896			4200	0,098523	98,5	21,93	32,31	OK	0,012	0,0909	OK	
7	19304,95	1413,491			6000	0,076592	76,6	14,96	46,15	OK	0,009	0,0909	OK	
6	16581,713	1070,802			4200	0,061634	61,6	10,28	32,31	OK	0,010	0,0909	OK	
P7	5062,594	507,166			2900	0,051358	51,4	5,21	22,31	OK	0,005	0,0909	OK	
P6M	10651,995	443,512			1500	0,046149	46,1	5,15	11,54	OK	0,023	0,0909	OK	
P6	6027,517	408,893			1500	0,040995	41,0	5,09	11,54	OK	0,014	0,0909	OK	
P5M	10161,6065	330,154			1500	0,035906	35,9	5,06	11,54	OK	0,028	0,0909	OK	
P5	6050,137	289,344			1500	0,030844	30,8	4,97	11,54	OK	0,019	0,0909	OK	
P4M	12354,0515	261,411			1500	0,02587	25,9	4,85	11,54	OK	0,042	0,0909	OK	
P4	6027,517	192,444			1500	0,021021	21,0	4,66	11,54	OK	0,027	0,0909	OK	
P3M	11473,2865	163,807			1500	0,016366	16,4	4,38	11,54	OK	0,056	0,0909	OK	
P3	7733,648	114,572			1500	0,011986	12,0	3,94	11,54	OK	0,048	0,0909	OK	
P2M	11572,084	84,539			1500	0,008051	8,1	3,34	11,54	OK	0,083	0,0909	OK	
P2	6338,249	50,268			1500	0,004714	4,7	2,87	11,54	OK	0,066	0,0909	OK	

Continued Table 5.11 P-delta control in the x direction for Model 1

X Direction							Storey Drift Check			P-Delta Stability Check			
Storey	Px	Vx	Cd	Ie	hsx	U2		Δ	Δ Max	Δ < Δ Max	θ	θ max	θ < θ max
	F3	F1			mm	m	mm	mm	mm	mm			
	kN	kN											
P1M	10832,672	32,017	5,5	1,5	1400	0,001847	1,8	1,29	10,77	OK	0,085	0,0909	OK
P1	9067,4965	10,280			1500	0,000559	0,6	0,56	11,54	OK	0,090	0,0909	OK
B	9534,489	0,000			1500	0	0,0	0,00	11,54	OK			

Table 5. 12 P-delta control in the y direction for Model 1

Y Direction							Storey Drift Check			P-Delta Stability Check			
Storey	Px	Vx	Cd	Ie	hsx	U2		Δ	Δ Max	Δ < Δ Max	θ	θ max	θ < θ max
	F3	F1			mm	m	mm	mm	mm	mm			
	kN	kN											
Rooftop	13505,099	3480,120	5,5	1,5	4200	0,132596	132,596	9,98	32,31	OK	0,0025	0,09091	OK
12	16626,247	3837,909			4200	0,122621	122,621	11,69	32,31	OK	0,0033	0,09091	OK
11	16626,247	3325,616			4200	0,110929	110,929	12,61	32,31	OK	0,0041	0,09091	OK
10	16626,247	2841,599			4200	0,098323	98,323	12,94	32,31	OK	0,0049	0,09091	OK
9	16626,247	2386,961			4200	0,085387	85,387	13,03	32,31	OK	0,0059	0,09091	OK
8	16626,247	2024,896			4200	0,072361	72,361	19,49	32,31	OK	0,0104	0,09091	OK

Continued Table 5.12 P-delta control in the y direction for Model 1

Storey	Px (kN)	Vx (kN)	Cd	Ie	hsx	U2		Δ	Δ Max	$\Delta < \Delta$ Max	θ	θ max	$\theta < \theta$ max
7	19304,95	1413,491	5,5	1,5	6000	0,05287	52,870	12,64	46,15	OK	0,0078	0,09091	OK
6	16581,713	1070,802			4200	0,040234	40,234	8,04	32,31	OK	0,0081	0,09091	OK
P7	5062,594	507,166			2900	0,032192	32,192	3,82	22,31	OK	0,0036	0,09091	OK
P6M	10651,995	443,512			1500	0,028376	28,376	3,43	11,54	OK	0,0150	0,09091	OK
P6	6027,517	408,893			1500	0,024949	24,949	3,40	11,54	OK	0,0091	0,09091	OK
P5M	10161,6065	330,154			1500	0,021546	21,546	3,10	11,54	OK	0,0173	0,09091	OK
P5	6050,137	289,344			1500	0,018451	18,451	3,13	11,54	OK	0,0119	0,09091	OK
P4M	12354,0515	261,411			1500	0,015324	15,324	2,75	11,54	OK	0,0236	0,09091	OK
P4	6027,517	192,444			1500	0,012575	12,575	2,72	11,54	OK	0,0155	0,09091	OK
P3M	11473,2865	163,807			1500	0,009857	9,857	2,44	11,54	OK	0,0311	0,09091	OK
P3	7733,648	114,572			1500	0,007415	7,415	2,35	11,54	OK	0,0288	0,09091	OK
P2M	11572,084	84,539			1500	0,005067	5,067	2,00	11,54	OK	0,0498	0,09091	OK
P2	6338,249	50,268			1500	0,003067	3,067	1,59	11,54	OK	0,0365	0,09091	OK
P1M	10832,672	32,017			1400	0,001477	1,477	1,08	10,77	OK	0,0709	0,09091	OK
P1	9067,4965	10,280			1500	0,000401	0,401	0,40	11,54	OK	0,0643	0,09091	OK
B	9534,489	0,000			1500	0	0,000	0,00	11,54	OK			

Based on the results presented in Table 5.11 and Table 5.12, there are no stability ratio values exceeding 0.1 on each floor of the building structure in Model 1. In accordance with the provisions outlined in SNI 1726:2019, the section related to the P-delta effect, the stability ratio is an important parameter used to evaluate the influence of the combination between vertical axial load (P) and the relative lateral displacement between floors (delta) on the overall stability of the structure. The threshold value of 0.1 established in SNI 1726:2019 indicates a safe limit where the P-delta effect is still considered small and insignificant to the structural performance.

If the stability ratio value on a floor is greater than 0.1, then the P-delta effect must be explicitly accounted for in the structural analysis, as it can cause significant increases in internal forces and deformations. This condition has the potential to reduce the stability of the structure and trigger dangerous geometric instability, especially when the structure experiences large lateral displacements due to seismic loads. However, based on the analysis results of Model 1, the stability ratio values across all floors remain below the threshold of 0.1, so the P-delta effect can be considered insignificant and no additional correction to the structural response is necessary.

The absence of stability ratio values exceeding this limit indicates that the building structure in Model 1 has a relatively stable behaviour against lateral displacements caused by earthquake loads. Thus, no special adjustments related to the P-delta effect are required in the design of the building's structural elements. Compliance with these provisions reflects the application of earthquake resilience planning principles in SNI 1726:2019, which aims to ensure that the structure has adequate capacity to withstand lateral forces without experiencing significant instability that could jeopardize the overall performance of the building.

In the structure of building model 1, there are no portal columns that are discontinuous from the roof to the pile cap, there is no void area exceeding 50% of the diaphragm area, and there are no bent portal columns, so the irregularities of the discontinuity of the vertical load-bearing vertical plane, the discontinuity of the

diaphragm, and the irregularity due to the perpendicular shift of the plane are ignored.

5.2.2 Results of Model 2 Structure Analysis

Model 2 is a building structure that employs a SRPMK system. In Model 2, the combination of modal mass participation reaches 100 percent (*Sum Ux* and *Sum Uy*) at mode 17, thus the mass participation has met the requirements. The data on mass participation can be seen in Table 5.13.

Table 5. 13 Data output period of Model 2 structural analysis program.

Mode	Period (sec)	Sum UX	Sum UY	Sum RZ
1	1,9015	0	0,7847	0
2	1,5025	0,0003	0,7847	0.7417
3	1,2264	0,6884	0,7847	0.7417
4	1,0951	0,6884	0,8947	0.7417
5	0,6480	0,6884	0,8947	0.8779
6	0,4728	0,6884	0,9367	0.8779
7	0,3839	0,8693	0,9367	0.8779
8	0,2836	0,8693	0,9367	0.9344
9	0,1931	0,8693	0,9619	0.9344
10	0,1806	0,8693	0,9778	0.9344
11	0,1613	0,9381	0,9778	0.9344
12	0,1514	0,9381	0,9778	0.9642
13	0,0931	0,9472	0,9931	0,97963
14	0,0894	0,9629	1	0,98716
15	0,0868	0,9787	1	0,98716
16	0,0837	0,9944	1	0,99372
17	0,0836	1	1	0,99373
18	0,0835	1	1	0,99373

Based on the results presented in Table 5.12, it can be observed that in the structure of Model 2, there is an increase in the natural vibration period value of the structure compared to Model 1, where the vibration period value increased from 1.719 seconds to 1.9015 seconds. Referring to the provisions in SNI 1726:2019, the natural vibration period of a structure is an important parameter in earthquake resilience planning because it reflects the dynamic characteristics and stiffness level of the building structure.

5.2.2.1 Story Shear

Story shear in the context of seismic analysis refers to the lateral force acting on each level of a structure due to seismic loads. When analyzed using the Static Equivalent approach as defined by SNI 1726:2019, this means that the seismic effects are approximated through an equivalent static lateral force that is calculated and distributed across each story of the building. Example of calculating the story shear at the 10th floor as follows:

$$C_{VX} = \frac{W_x h_x^k}{\sum_{i=1}^n W_i h_i^k}$$

Where :

C_{VX}	= Faktor vertical distribution
V_x	= gaya lateral desain total
W_i or W_x	= Wight of floor
h_i or h_x	= Height of loor
K	= The exponent associated with the structural period

is defined as follows :

For structures with $T \leq 0,5$ seconds, $k = 1$

For structures with $T \geq 2,5$ seconds, $k = 2$

For structures with $0,5 < T < 2,5$ seconds, $k = 2$ or determined through linear interpolation between 1 and 2.

10th floor data :

T	= 1,901 s
K	= 1,700 from linear interpolation between

$$\begin{aligned}
H_{10} &= 40,9 \text{ m} \\
W_{10} &= 16626,247 \text{ kN} \\
W_{10}h_{10}^k &= 16626,247 \times 40,9^{1,700} \\
&= 9160784,022 \text{ kNm} \\
\sum_{i=1}^n Wih_i^k &= 73417707,68 \text{ kNm} \\
C_{V10} &= \frac{9160784,022 \text{ kNm}}{73417707,68 \text{ kNm}} \\
&= 0,124 \\
V_{10} &= 0,124 + 0,478 \\
&= 0,603
\end{aligned}$$

The control of Story shear for Model 2 is presented in Tables 5.14.

Table 5. 14 Story shear for Model 2

Storey	Weight of floor (KN)	H (m)	K	wi.hi ^k	Cvx	Vx	Vi/Vi+1
Rooftop	13505,099	53,5	1,7008	11748852,917	0,160	0,160	
12	16626,247	49,3	1,7008	12586460,757	0,171	0,331	2,071
11	16626,247	45,1	1,7008	10817701,206	0,147	0,479	1,445
10	16626,247	40,9	1,7008	9160784,022	0,125	0,604	1,261
9	16626,247	36,7	1,7008	7619036,198	0,104	0,707	1,172
8	16626,247	32,5	1,7008	6196263,160	0,084	0,792	1,119
7	19304,95	26,5	1,7008	5084568,484	0,069	0,861	1,087
6	16581,713	22,3	1,7008	3256558,543	0,044	0,905	1,052
P7	6117,056	19,4	1,7008	947919,044	0,013	0,918	1,014
P6M	11037,136	17,9	1,7008	1491578,746	0,020	0,939	1,022
P6	6117,056	16,4	1,7008	712341,311	0,010	0,948	1,010
P5M	10523,9185	14,9	1,7008	1041055,834	0,014	0,962	1,015
P5	6117,056	13,4	1,7008	505203,025	0,0069	0,969	1,007
P4M	11892,9355	11,9	1,7008	802648,795	0,0109	0,980	1,011
P4	6117,056	10,4	1,7008	328293,216	0,0045	0,985	1,005
P3M	11892,9355	8,9	1,7008	489738,354	0,0067	0,991	1,007
P3	6117,056	7,4	1,7008	184029,863	0,0025	0,994	1,003
P2M	12151,38	5,9	1,7008	248686,472	0,0034	0,997	1,003
P2	6810,487	4,4	1,7008	84631,163	0,0012	0,998	1,001
P1M	14211,701	3	1,7008	92068,281	0,0013	0,9997	1,001
P1	9678,4669	1,5	1,7008	19288,292	0,0003	1,0	1,000
B	10905,369	0	1,7008	0	0	1	1,000
Total	262212,6064			73417707,68	1		

5.2.2.2 Soft Story Stiffness Irregularity

Based on SNI 1726:2019, a soft story is defined as a level where its lateral stiffness is less than 70 percent of the lateral stiffness of the level above it or less than 80 percent of the average lateral stiffness of the three levels above it. For this purpose, it is necessary to examine the building's structure to determine whether it has a soft-story irregularity or not. Example of calculating the structural stiffness at the 10th floor due to an earthquake load in the x-direction is as follows :

Structural stiffness conditions :

$$K_i > 0,7 K_{i+1} \rightarrow \text{Regular}$$

$$K_i > 0,8 \text{ Avg}(K_{i+1} + K_{i+2} + K_{i+3}) \rightarrow \text{Regular}$$

Where :

$$\Delta i = U_{10} - U_9 = 0,015258 - 0,14005 = 0,012791 \text{ m}$$

$$K_i = \frac{V_x}{\Delta i} = \frac{4}{0,012791} = 312,72 \text{ Kn/m}$$

$$K_{i+1} = k_{i11} = 263,551 \text{ Kn/m}$$

$$312,72 > 0,7 \times 263,551$$

$$312,72 > 184,485 \rightarrow \text{Regular}$$

Explanation:

$$Z = \text{Height of floor (mm)}$$

$$F_x = \text{Vertical load on and level x (kN)}$$

V_x = Total vertical load at level x (kN)

Δ_i = Drift at level x (m)

The control of soft story irregularity in the x and y directions for Model 2 is presented in Tables 5.15 and 5.16.

Table 5. 15 Soft story stiffness irregularity control in x-direction earthquake.

Storey	Z	FX	Akumulatif	U1	Δ_i	K	Cek	Conclusion	cek	Conclusion
	(mm)	(kN)	V_x (kN)	mm	(m)	(Kn/mm)	70%Ki+1		80%(avgKi+1)	
Rooftop	52000	1	1	0,181	0,007	127,665	0	Regular	0	Regular
12	47800	1	2	0,174	0,009	204,352	89,365	Regular	102,132	Regular
11	43600	1	3	0,164	0,011	263,550	143,046	Regular	132,807	Regular
10	39400	1	4	0,152	0,012	312,719	184,485	Regular	158,818	Regular
9	35200	1	5	0,140	0,014	352,708	218,903	Regular	208,166	Regular
8	31000	1	6	0,125	0,023	253,410	246,896	Regular	247,727	Regular
7	25000	1	7	0,102	0,016	418,135	177,387	Regular	245,023	Regular
6	20800	1	8	0,085	0,012	640,102	292,694	Regular	273,134	Regular
P7	17900	1	9	0,072	0,006	1308,14	448,071	Regular	349,772	Regular
P6M	16400	1	10	0,066	0,006	1435,75	915,697	Regular	631,033	Regular
P6	14900	1	11	0,059	0,007	1568,068	1005,025	Regular	902,397	Regular
P5M	13400	1	12	0,052	0,007	1709,889	1097,647	Regular	1149,855	Regular
P5	11900	1	13	0,045	0,006	1862,731	1196,922	Regular	1256,988	Regular
P4M	10400	1	14	0,038	0,006	2033,997	1303,911	Regular	1370,850	Regular
P4	8900	1	15	0,031	0,006	2239,14	1423,797	Regular	1495,097	Regular
P3M	7400	1	16	0,024	0,006	2506,658	1567,398	Regular	1636,231	Regular

Continued Table 5.15 Soft story stiffness irregularity control in x-direction earthquake.

Storey	Z	FX	Akumulatif	U1	Δi	K	Cek	Conclusion	cek	Conclusion
	(mm)	(kN)	V _x (kN)	mm	(m)	(Kn/mm)	70%Ki+1		80%(avgKi+1)	
P3	5900	1	17	0,018	0,005	2899,539	1754,660	Regular	1807,945	Regular
P2M	4400	1	18	0,012	0,005	3561,535	2029,677	Regular	2038,756	Regular
P2	2900	1	19	0,007	0,003	5212,62	2493,074	Regular	2391,395	Regular
P1M	1500	1	20	0,003	0,002	7770,008	3648,834	Regular	3112,985	Regular
P1	0	1	21	0,0009	0,0009	21105,53	5439,005	Regular	4411,776	Regular
B	-1500	0	0	0	0		0			

Table 5. 16 Soft story stiffness irregularity control in y-direction earthquake.

Storey	Z	FX	Akumulatif	U2	Δi	K	Cek	Conclusion	cek	Conclusion
	(mm)	(kN)	V _x (kN)	m	(m)	(Kn/m)	70%Ki+1		80%(avgKi+1)	
Rooftop	52000	1	1	0,135	0,008	117,481	0	Regular	0	Regular
12	47800	1	2	0,127	0,010	191,168	82,236	Regular	93,984	Regular
11	43600	1	3	0,117	0,011	251,487	133,817	Regular	123,459	Regular
10	39400	1	4	0,105	0,012	314,243	176,041	Regular	149,369	Regular
9	35200	1	5	0,092	0,013	375,968	219,970	Regular	201,839	Regular
8	31000	1	6	0,079	0,020	290,796	263,177	Regular	251,119	Regular
7	25000	1	7	0,058	0,012	543,436	203,557	Regular	261,601	Regular
6	20800	1	8	0,045	0,008	936,658	380,405	Regular	322,720	Regular
P7	17900	1	9	0,036	0,004	2090,106	655,660	Regular	472,237	Regular
P6M	16400	1	10	0,032	0,004	2428,363	1463,074	Regular	952,053	Regular
P6	14900	1	11	0,028	0,003	2791,878	1699,854	Regular	1454,700	Regular

Continued Table 5.16 Soft story stiffness irregularity control in y-direction earthquake.

Storey	Z	FX	Akumulatif	U2	Δi	K	Cek	Conclusion	cek	Conclusion
	(mm)	(kN)	V _x (kN)	m	(m)	(Kn/m)	70%Ki+1		80%(avgKi+1)	
P5M	13400	1	12	0,024	0,003	3192,338	1954,314	Regular	1949,426	Regular
P5	11900	1	13	0,020	0,003	3641,456	2234,636	Regular	2243,354	Regular
P4M	10400	1	14	0,017	0,003	4161,712	2549,0196	Regular	2566,846	Regular
P4	8900	1	15	0,013	0,003	4778,591	2913,198	Regular	2932,135	Regular
P3M	7400	1	16	0,010	0,002	5540,166	3345,014	Regular	3355,136	Regular
P3	5900	1	17	0,007	0,002	6604,506	3878,116	Regular	3861,458	Regular
P2M	4400	1	18	0,005	0,002	8222,932	4623,154	Regular	4512,870	Regular
P2	2900	1	19	0,003	0,001	11949,685	5756,052	Regular	5431,361	Regular
P1M	1500	1	20	0,001	0,001	17777,777	8364,779	Regular	7140,566	Regular
P1	0	1	21	0,0004	0,0004	49180,327	12444,444	Regular	10120,105	Regular
B	-1500	0	0	0	0	0				

Based on Tables 5.15 and 5.16, in Model 2, there are no floors with lateral stiffness less than 70% compared to the floor above or less than 80% of the average of the three floors above, both in the x and y directions. Thus, Model 2 does not exhibit a soft-story irregularity, a result that is also the same as Model 1. According to SNI 1726:2019, weak-level irregularity occurs when the lateral stiffness of a floor decreases drastically. The absence of this irregularity indicates a uniform distribution of stiffness, ensuring the structure is stable and safe against seismic loads.

5.2.2.3 Mass Irregularity

According to SNI 1726:2019, a mass irregularity is defined to exist if there is a level with an effective mass greater than 150 percent of the effective mass of the nearby level. Therefore, it is necessary to examine the building structure to determine whether it has mass irregularities or not. Example of mass irregularity calculation on the 10th floor is as follows:

Condition of Mass Irregularity:

$W_i > 150\% W_{i+1} \rightarrow \text{Irregular}$

$W_{i_{10}} > 150\% W_{i_{11}} \rightarrow 16626,247 < 24939,3705 \rightarrow \text{Regular}$

The control of mass irregularity for Model 2 can be seen in Table 5.17.

Table 5. 17 Mass irregularity control for Model 2

Storey	Weight of floor (KN)	150%Wi+1	Conclusion
Rooftop	13505,099	0	
12	16626,247	20257,649	Regular
11	16626,247	24939,371	Regular
10	16626,247	24939,371	Regular
9	16626,247	24939,371	Regular
8	16626,247	24939,371	Regular
7	19304,95	24939,371	Regular
6	16581,713	28957,425	Regular
P7	6117,056	24872,57	Regular
P6M	11037,136	9175,584	Irregular
P6	6117,056	16555,704	Regular
P5M	10523,9185	9175,584	Irregular
P5	6117,056	15785,878	Regular
P4M	11892,9355	9175,584	Irregular
P4	6117,056	17839,403	Regular
P3M	11892,9355	9175,584	Irregular
P3	6117,056	17839,403	Regular
P2M	12151,38	9175,584	Irregular
P2	6810,487	18227,07	Regular
P1M	14211,701	10215,731	Irregular
P1	9678,4669	21317,552	Regular
B	10905,369	14517,7	Regular
Total	262212,6064		

Based on Table 5.17, there is an effective mass percentage exceeding 150% of the floor above it, particularly on the P1M, P3M, P4M, P5M, and P6M floors, indicating that the building structure in Model 2 experiences mass irregularity. This irregularity occurs due to a significant increase in mass on those floors compared to the floor above, which can affect the overall mass distribution in the structure. However, according to SNI 1726:2019, if there is no inter-floor drift ratio greater than 1.3 times compared to the drift ratio of the floor above it, then vertical irregularity Type 2 b (mass irregularity) can be considered insignificant and can be disregarded.

Nevertheless, this calculation must still consider the influence of mass irregularity on the structural response to seismic loads. In the case of Model 2, despite the mass irregularity, the building structure still meets the SNI 1726:2019 requirements because the inter-story drift ratio does not exceed the permitted limit. Additionally, the total weight of the building in Model 2 experienced a slight increase from 259,637.658 kN to 262,212,606.4 kN. This weight increase is caused by the adjustment of the main beam dimensions on the parking floor. This increase in dimensions contributes to the overall mass of the building, but does not significantly affect the structural performance, considering that the inter-story drift remains within the safe limits specified in SNI 1726:2019.

5.2.2.4 Geometric Irregularities

Based on SNI 1726:2019, vertical geometric irregularity in building structures is defined as a condition where the horizontal dimensions of the seismic force-resisting system at each level exceed 130 percent. Example of calculating geometric irregularity on the 10th floor is as follows:

$$D_i > 130\% D_{i+1} \rightarrow \text{Irregular}$$

$$D_{i10} > 130\% D_{i11} \rightarrow 43,6 \text{ m} < 56,68 \text{ m} \rightarrow \text{Regular}$$

Explanation:

D_i = The width of the earthquake-resistant portal from the first column to the farthest column end.

The control of geometric irregularity in the x and y directions for Model 2 is presented in Tables 5.18 and 5.19.

Table 5. 18 Geometric Irregularities control in x-direction earthquake

Storey	Di (m)	di/di+1	Conclusion
Rooftop	43,6	0	
12	43,6	56,68	Regular
11	43,6	56,68	Regular
10	43,6	56,68	Regular
9	43,6	56,68	Regular
8	43,6	56,68	Regular
7	43,6	56,68	Regular
6	43,6	56,68	Regular
P7	43,6	56,68	Regular
P6M	43,6	56,68	Regular
P6	43,6	56,68	Regular
P5M	43,6	56,68	Regular
P5	43,6	56,68	Regular
P4M	43,6	56,68	Regular
P4	43,6	56,68	Regular
P3M	43,6	56,68	Regular
P3	43,6	56,68	Regular
P2M	43,6	56,68	Regular
P2	43,6	56,68	Regular
P1M	43,6	56,68	Regular
P1	43,6	56,68	Regular
B	43,6	56,68	Regular

Table 5. 19 Geometric Irregularities control in y-direction earthquake

Storey	Di (m)	di/di+1	Conclusion
Rooftop	24,792	0	
12	24,792	32,2296	Regular
11	24,792	32,2296	Regular
10	24,792	32,2296	Regular
9	24,792	32,2296	Regular
8	24,792	32,2296	Regular
7	24,792	32,2296	Regular
6	24,792	32,2296	Regular
P7	24,792	32,2296	Regular

Continued Table 5.19 Geometric Irregularities control in y-direction earthquake

Storey	Di (m)	di/di+1	Conclusion
P6M	24,792	32,2296	Regular
P6	24,792	32,2296	Regular
P5M	24,792	32,2296	Regular
P5	24,792	32,2296	Regular
P4M	24,792	32,2296	Regular
P4	24,792	32,2296	Regular
P3M	24,792	32,2296	Regular
P3	24,792	32,2296	Regular
P2M	24,792	32,2296	Regular
P2	24,792	32,2296	Regular
P1M	24,792	32,2296	Regular
P1	24,792	32,2296	Regular
B	24,792	32,2296	Regular

Based on Table 5.18 and Table 5.19, no vertical geometric irregularities were found on each floor of the building structure in Model 2. Vertical geometric irregularity is defined in SNI 1726:2019 as a condition where the horizontal dimensions of the seismic resistance system on each floor exceed 130 percent of the dimensions of the floor below it. In other words, if there are significant changes in the horizontal dimensions between floors, this can affect the distribution of lateral forces during an earthquake, which risks reducing the structural performance.

However, in Model 2, the distribution of horizontal dimensions between floors remains consistent, which means that each floor complies with the provisions in SNI 1726:2019 regarding vertical geometric irregularities. The absence of this geometric irregularity ensures that the building structure has a more uniform response to seismic loads, with stiffness well-distributed throughout the building's height, thereby reducing the potential for stability issues during an earthquake event.

5.2.2.5 Story Drift and Drift Ratio

The inter-floor displacement is the difference in the deflection value of the building between floor i and the floor below it. Based on the SNI 1726:2019 regulations, the inter-floor displacement control only has one performance limit, which is the ultimate limit state. The design-level inter-floor displacement must not exceed the allowable inter-floor displacement multiplied by the redundancy factor. Story drift and drift ratio for the 10th floor can be calculated using the following equation:

$$\frac{\delta_{Max}}{\delta_{Average}} < 1,2 \rightarrow \text{Regular}$$

$$\begin{aligned}\delta A_{10} &= \text{Def. Joint } Ax_{10} - \text{Def. Joint } Ax_9 \\ &= 0,130697 - 0,118531 \\ &= 0,012166 \\ \delta B_{10} &= \text{Def. Joint } Bx_{10} - \text{Def. Joint } Bx_9 \\ &= 0,152865 - 0,140067 \\ &= 0,012798 \\ \delta_{Max} &= 0,012798 \\ \delta_{Average} &= 0,012482 \\ \frac{\delta_{Max}}{\delta_{Average}} &= \frac{0,012798}{0,012482} \\ &= 1,02531 \\ &= 1,025 < 1,2 \rightarrow \text{Regular}\end{aligned}$$

Where :

δ = Story drift (m)

Tables 5.19 and 5.20 present the results of the deviation values between floors for Model 2

Table 5. 20 Story Drift and Drift Ratio X – direction

Storey	Def. Joint Ax	δA	Def. Joint Bx	δB	δ Max	δ Average	Rasio δ Max / δ Average	Conclusion	Ax	Ax used	5% Ax
Rooftop	0,1584	0,007482	0,181	0,0078	0,0078	0,0076	1,022	Regular	0,726	1	0,05
12	0,1510	0,009372	0,174	0,0097	0,0097	0,0095	1,021	Regular	0,724	1	0,05
11	0,1416	0,010932	0,164	0,0113	0,0113	0,0111	1,019	Regular	0,722	1	0,05
10	0,1306	0,012166	0,152	0,0127	0,0127	0,0124	1,025	Regular	0,730	1	0,05
9	0,1185	0,013405	0,140	0,0141	0,0141	0,0137	1,027	Regular	0,733	1	0,05
8	0,1051	0,02175	0,125	0,0236	0,0236	0,0227	1,042	Regular	0,754	1	0,05
7	0,0833	0,015117	0,102	0,0167	0,0167	0,0159	1,051	Regular	0,768	1	0,05
6	0,0682	0,010701	0,085	0,0124	0,0124	0,0115	1,077	Regular	0,806	1	0,05
P7	0,0575	0,005527	0,072	0,0069	0,0069	0,0062	1,111	Regular	0,857	1	0,05
P6M	0,0520	0,005588	0,066	0,0069	0,0069	0,0062	1,108	Regular	0,853	1	0,05
P6	0,0464	0,00558	0,059	0,0070	0,0070	0,0063	1,114	Regular	0,862	1	0,05
P5M	0,0408	0,005596	0,052	0,0070	0,0070	0,0063	1,111	Regular	0,858	1	0,05
P5	0,0352	0,005553	0,045	0,0069	0,0069	0,0062	1,113	Regular	0,861	1	0,05
P4M	0,0297	0,005441	0,038	0,0068	0,0068	0,0061	1,116	Regular	0,865	1	0,05
P4	0,0242	0,005291	0,031	0,0066	0,0066	0,005	1,117	Regular	0,867	1	0,05
P3M	0,0189	0,005009	0,024	0,0063	0,0063	0,0056	1,120	Regular	0,871	1	0,05
P3	0,0139	0,004546	0,0181	0,0058	0,0058	0,0052	1,126	Regular	0,880	1	0,05
P2M	0,0094	0,00388	0,0122	0,0050	0,0050	0,0044	1,133	Regular	0,892	1	0,05
P2	0,0055	0,002795	0,0071	0,0036	0,0036	0,0032	1,131	Regular	0,888	1	0,05

Continued Table 5.20 Story Drift and Drift Ratio X – direction

Storey	Def. Joint Ax	δA	Def. Joint Bx	δB	δ Max	δ Average	Rasio δ Max / δ Average	Conclusion	Ax	Ax used	5% Ax
P1M	0,0027	0,00200	0,0035	0,0026	0,0026	0,00230	1,130	Regular	0,887	1	0,05
P1	0,0007	0,00074	0,0009	0,0009	0,0009	0,00085	1,122	Regular	0,874	1	0,05
B		0	0	0	0	0					

Table 5. 21 Story Drift and Drift Ratio Y – direction.

Storey	Def. Joint Ay	δA	Def. Joint By	δB	δ Max	δ Average	Rasio δ Max / δ Average	Conclusion	Ax	Ax used	5% Ax	Ey
Rooftop	0,12340	0,00911	0,1337	0,0097	0,0097	0,0094	1,0359240	Regular	0,745	1	0,0500	0,939
12	0,1142	0,01073	0,1239	0,0113	0,0113	0,0110	1,0300921	Regular	0,736	1	0,0500	0,928
11	0,1035	0,01158	0,1125	0,0122	0,0122	0,0119	1,0289162	Regular	0,735	1	0,0500	0,926
10	0,0919	0,01190	0,1002	0,0126	0,0126	0,0122	1,0303721	Regular	0,737	1	0,0500	0,929
9	0,0800	0,01202	0,0875	0,0127	0,0127	0,0124	1,0311480	Regular	0,738	1	0,0500	0,930
8	0,0680	0,01802	0,0747	0,0190	0,0190	0,0185	1,0285113	Regular	0,734	1	0,0500	0,925
7	0,0500	0,01177	0,0557	0,0123	0,0123	0,0120	1,0235084	Regular	0,727	1	0,0500	0,916
6	0,0382	0,00787	0,0433	0,0081	0,0081	0,0080	1,0169068	Regular	0,718	1	0,0500	0,904
P7	0,0303	0,00311	0,0352	0,0041	0,0041	0,0036	1,1369559	Regular	0,897	1	0,0500	1,131

Continued Table 5.21 Story Drift and Drift Ratio Y – direction

Storey	Def. Joint Ay	δA	Def. Joint By	δB	δ Max	δ Average	Rasio δ_{Max} / δ Average	Conclusion	Ax	Ax used	5% Ax	Ey
P6M	0,02725	0,00328	0,0311	0,0038	0,0038	0,0035	1,0782	Regular	0,807	1	0,0500	1,017
P6	0,0239	0,00298	0,0272	0,0037	0,0037	0,0033	1,1208	Regular	0,872	1	0,0500	1,099
P5M	0,0209	0,00311	0,0234	0,0035	0,0035	0,0033	1,0691	Regular	0,793	1	0,0500	1,000
P5	0,0178	0,00278	0,0199	0,0034	0,0034	0,0031	1,1121	Regular	0,858	1	0,0500	1,082
P4M	0,0151	0,00284	0,0164	0,0031	0,0031	0,0030	1,0564	Regular	0,775	1	0,0500	0,976
P4	0,0122	0,00251	0,0132	0,0030	0,0030	0,0027	1,0888	Regular	0,823	1	0,0500	1,037
P3M	0,0097	0,00250	0,0102	0,0027	0,0027	0,0026	1,0441	Regular	0,757	1	0,0500	0,954
P3	0,0072	0,00208	0,007	0,0024	0,0024	0,0022	1,0814	Regular	0,812	1	0,0500	1,023
P2M	0,0051	0,00191	0,0050	0,0020	0,0020	0,0019	1,0393	Regular	0,750	1	0,0500	0,945
P2	0,0032	0,00163	0,0029	0,0015	0,0016	0,0015	1,0309	Regular	0,738	1	0,0500	0,930
P1M	0,0016	0,00119	0,0014	0,0010	0,0011	0,0011	1,0505	Regular	0,766	1	0,0500	0,965
P1	0,0004	0,00041	0,0003	0,0003	0,0004	0,0003	1,0765	Regular	0,804	1	0,0500	1,014
B	0	0	0	0	0	0						

Based on Table 5.19 and Table 5.20, it can be observed that there is no torsional irregularity between the floors of the building structure in Model 2, which is defined as a condition where the ratio between the maximum displacement and the average displacement is greater than 1.2. This indicates that Model 2 does not exhibit significant torsional irregularity, which could affect the distribution of seismic forces and cause uneven rotation in the building structure. These results are consistent with those found in Model 1, which also shows a displacement ratio that meets the requirements, without any torsional irregularities exceeding the allowed threshold.

However, although torsional irregularity does not occur in Model 2, there is an increase in inter-story drift compared to Model 1. This increase is caused by the reduction in structural stiffness in Model 2, which can result in greater lateral movement on each floor, although the torsional irregularity can still be ignored. According to SNI 1726:2019, torsional irregularity occurs if the ratio of maximum displacement to average displacement exceeds 1.2, which can cause significant changes in force distribution and affect the overall stability of the building.

Overall, although Model 2 does not show torsional irregularity, the decrease in stiffness that occurs still needs to be considered, as it can affect the structural response to lateral loads due to an earthquake. Nevertheless, by maintaining a displacement ratio that meets the standards, Model 2 still demonstrates compliance with the principles outlined in SNI 1726:2019 regarding torsional irregularity.

5.2.2.6 Internal Angle Irregularity

Based on SNI 1726:2019, The internal angle irregularity is defined to be present if both projections of the internal angle structure plan are greater than 15 percent of the dimension of areas of concern. Structure plan in the specified direction can be seen on figure 5.6.

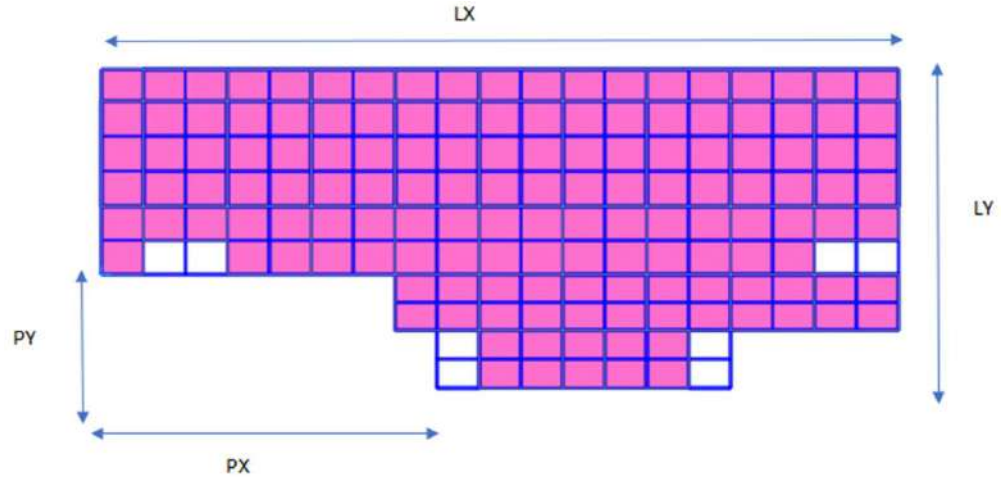


Figure 5. 2 Internal angular irregularity

Example of calculating internal angular irregularity on the 10th floor is as follows:

Condition of Mass Irregularity:

$P_x > 15\% L_x \rightarrow \text{Irregular}$

$32000 > 15\% 76000$

$32000 > 11400 \rightarrow \text{Irregular}$

$P_y > 15\% L_y \rightarrow \text{Irregular}$

$30300 > 15\% 10800$

$30300 > 4545 \rightarrow \text{Irregular}$

The control of internal angular irregularity for Model 2 is presented in Tables 5.22.

Table 5. 22 Internal angle irregularity

Storey	LX (mm)	PX (mm)	15%LX	STATUS	LY (mm)	PY (mm)	15%LY	STATUS
Rooftop	76000	32000	11400	Irregular	30300	10800	4545	Irregular
12	76000	32000	11400	Irregular	30300	10800	4545	Irregular
11	76000	32000	11400	Irregular	30300	10800	4545	Irregular
10	76000	32000	11400	Irregular	30300	10800	4545	Irregular

Continued Table 5.22 Internal angle irregularity

Storey	LX (mm)	PX (mm)	15%LX	STATUS	LY (mm)	PY (mm)	15%LY	STATUS
9	76000	32000	11400	Irregular	30300	10800	4545	Irregular
8	76000	32000	11400	Irregular	30300	10800	4545	Irregular
7	76000	32000	11400	Irregular	30300	10800	4545	Irregular
6	76000	32000	11400	Irregular	30300	10800	4545	Irregular
P7	76000	32000	11400	Irregular	30300	10800	4545	Irregular
P6M	76000	36000	11400	Irregular	14400	12000	2160	Irregular
P6	48000	16000	7200	Irregular	14400	12000	2160	Irregular
P5M	76000	36000	11400	Irregular	19500	1000	2925	Irregular
P5	48000	16000	7200	Irregular	14400	12000	2160	Irregular
P4M	76000	32000	11400	Irregular	37500	18000	5625	Irregular
P4	48000	16000	7200	Irregular	14400	12000	2160	Irregular
P3M	76000	32000	11400	Irregular	37500	18000	5625	Irregular
P3	48000	16000	7200	Irregular	14400	12000	2160	Irregular
P2M	76000	32000	11400	Irregular	37500	18000	5625	Irregular
P2	48000	16000	7200	Irregular	14400	12000	2160	Irregular
P1M	76000	32000	11400	Irregular	37500	18000	5625	Irregular
P1	48000	16000	7200	Irregular	14400	12000	2160	Irregular
B	76000	32000	11400	Irregular	37500	18000	5625	Irregular

Based on the results presented in Table 5.21, it is known that there is a vertical irregularity in all floors of the building structure in Model 2. Referring to the provisions set forth in SNI 1726:2019, if there is a type 2 horizontal irregularity, characterized by a significant change in the distribution of stiffness or the horizontal geometry of the structure between floors, then structural elements such as diaphragms, chord elements, collectors, and their connections are required to be designed with an additional multiplier factor of 1.25. The increase in this factor aims to ensure that the capacity of these elements to transfer lateral forces due to an earthquake can be enhanced, thereby compensating for the horizontal irregularity that occurs and preventing potential excessive stress concentration on these elements. This provision is an effort to maintain the optimal performance of the

structure in facing lateral loads that occur during an earthquake event, as well as to minimize the risk of partial collapse due to the weakness of diaphragm elements and their components. Thus, the application of this multiplier factor is in accordance with the earthquake resilience planning principles outlined in SNI 1726:2019 to enhance the overall reliability and safety of the building structure.

5.2.2.7 Influence of P-Delta Effect

Based on SNI 1726:2019, the P-delta effect must be considered for building structures that bear earthquake loads. However, the P-delta effect can be disregarded if the stability ratio is less than 0.1. The P-delta effect at the reviewed level must be calculated using the following equation:

$$\theta = \frac{P_x \Delta i_e}{V_x h_{sx} C_d}$$

$$\theta_{10} = \frac{P_{x10} \Delta_{10} i_e}{V_{x10} h_{sx10} C_d}$$

$$= \frac{1626,274 \times 12,17 \times 1,5}{2582,200 \times 4200 \times ,5} = 0,005 < 0,1 \rightarrow \text{ok}$$

Explanation :

C_d = deflation amplification factor

Δ_x = drift at level x (mm)

I_e = earthquake priority factor

P_x = Total vertical design load at and above level x (kN)

V_x = high level below level x (kN)

The control of the P-delta influence in the X and Y directions can be seen in Tables 5.22 and 5.23.

Table 5. 23 P-delta control in the x direction for Model 2

X Direction							Storey Drift Check			P-Delta Stability Check			
Storey	P _x	V _x	Cd	I _e	h _{sx}	U1		Δ	Δ Max	Δ < Δ Max	θ	θ max	θ < θ max
	F3	F1											
	kN	kN			mm	m	mm	mm	mm	mm			
Rooftop	13505,099	3337,477	5,5	1,5	4200	0,1585	158,5	7,48	32,31	OK	0,002	0,091	OK
12	16626,247	3575,414			4200	0,1510	151,0	9,37	32,31	OK	0,003	0,091	OK
11	16626,247	3072,966			4200	0,1416	141,6	10,93	32,31	OK	0,004	0,091	OK
10	16626,247	2602,288			4200	0,1307	130,7	12,17	32,31	OK	0,005	0,091	OK
9	16626,247	2164,327			4200	0,1185	118,5	13,41	32,31	OK	0,007	0,091	OK
8	16626,247	1760,162			4200	0,1051	105,1	21,75	32,31	OK	0,013	0,091	OK
7	19304,95	1444,365			6000	0,0834	83,4	15,12	46,15	OK	0,009	0,091	OK
6	16581,713	925,085			4200	0,0683	68,3	10,70	32,31	OK	0,012	0,091	OK
P7	6117,056	269,274			2900	0,0576	57,6	5,53	22,31	OK	0,012	0,091	OK
P6M	11037,136	423,710			1500	0,0520	52,0	5,59	11,54	OK	0,026	0,091	OK
P6	6117,056	202,354			1500	0,0464	46,4	5,58	11,54	OK	0,031	0,091	OK
P5M	10523,9185	295,731			1500	0,0409	40,9	5,60	11,54	OK	0,036	0,091	OK
P5	6117,056	143,512			1500	0,0353	35,3	5,55	11,54	OK	0,043	0,091	OK
P4M	11892,9355	228,007			1500	0,0297	29,7	5,44	11,54	OK	0,052	0,091	OK
P4	6117,056	93,258			1500	0,0243	24,3	5,29	11,54	OK	0,063	0,091	OK
P3M	11892,9355	139,119			1500	0,0190	19,0	5,01	11,54	OK	0,078	0,091	OK
P3	6117,056	52,277			1500	0,0140	14,0	4,55	11,54	OK	0,097	0,091	Not OK
P2M	12151,38	70,644			1500	0,0094	9,4	3,88	11,54	OK	0,121	0,091	Not OK
P2	6810,487	24,041			1500	0,0055	5,5	2,80	11,54	OK	0,144	0,091	Not OK

Continued Table 5.23 P-delta control in the x direction for Model 2

X Direction							Storey Drift Check			P-Delta Stability Check			
Storey	Px	Vx	Cd	Ie	hsx	U2		Δ	Δ Max	Δ < Δ Max	θ	θ max	θ < θ max
	F3	F1			mm	m	mm	mm	mm	mm			
	kN	kN											
P1M	14211,701	26,154	5,5	1,5	1400	0,0028	2,8	2,00	10,77	OK	0,212	0,091	Not OK
P1	9678,4669	5,479			1500	0,0007	0,7	0,75	11,54	OK	0,241	0,091	Not OK
B	10905,369	0,000			1500	0,0000	0,0	0,00	11,54	OK			

Table 5. 24 P-delta control in the y direction for Model 2

Y Direction							Storey Drift Check			P-Delta Stability Check			
Storey	Px	Vx	Cd	Ie	hsx	U2		Δ	Δ Max	Δ < Δ Max	θ	θ max	θ < θ max
	F3	F1			mm	m	mm	mm	mm	mm			
	kN	kN											
Rooftop	13505,099	3337,477	5,5	1,5	4200	0,123409	123,409	9,11	32,31	OK	0,0024	0,091	OK
12	16626,247	3575,414			4200	0,114298	114,298	10,73	32,31	OK	0,0032	0,091	OK
11	16626,247	3072,966			4200	0,103565	103,565	11,59	32,31	OK	0,0041	0,091	OK
10	16626,247	2602,288			4200	0,091979	91,979	11,91	32,31	OK	0,0049	0,091	OK
9	16626,247	2164,327			4200	0,080071	80,071	12,02	32,31	OK	0,0060	0,091	OK
8	16626,247	1760,162			4200	0,072361	72,361	19,49	32,31	OK	0,0104	0,09091	OK

Continued Table 5.24 P-delta control in the y direction for Model 2

Storey	Px (kN)	Vx (kN)	Cd	Ie	hsx	U2		Δ	$\frac{\Delta}{Max}$	$\frac{\Delta}{\Delta Max}$	θ	θ_{max}	$\theta < \theta_{max}$
7	19304,95	1444,365	5,5	1,5	6000	0,050024	50,024	11,78	46,15	OK	0,0072	0,091	OK
6	16581,713	925,085			4200	0,038248	38,248	7,88	32,31	OK	0,0092	0,091	OK
P7	6117,056	269,274			2900	0,030369	30,369	3,11	22,31	OK	0,0067	0,091	OK
P6M	11037,136	423,710			1500	0,027256	27,256	3,28	11,54	OK	0,0155	0,091	OK
P6	6117,056	202,354			1500	0,023976	23,976	2,98	11,54	OK	0,0164	0,091	OK
P5M	10523,9185	295,731			1500	0,020996	20,996	3,11	11,54	OK	0,0201	0,091	OK
P5	6117,056	143,512			1500	0,017886	17,886	2,78	11,54	OK	0,0216	0,091	OK
P4M	11892,9355	228,007			1500	0,015104	15,104	2,84	11,54	OK	0,0269	0,091	OK
P4	6117,056	93,258			1500	0,012264	12,264	2,51	11,54	OK	0,0300	0,091	OK
P3M	11892,9355	139,119			1500	0,009751	9,751	2,51	11,54	OK	0,0390	0,091	OK
P3	6117,056	52,277			1500	0,007242	7,242	2,09	11,54	OK	0,0444	0,091	OK
P2M	12151,38	70,644			1500	0,005157	5,157	1,92	11,54	OK	0,0599	0,091	OK
P2	6810,487	24,041			1500	0,003241	3,241	1,63	11,54	OK	0,0841	0,091	OK
P1M	14211,701	26,154			1400	0,001609	1,609	1,19	10,77	OK	0,1264	0,091	Not OK
P1	9678,4669	5,479			1500	0,000415	0,415	0,42	11,54	OK	0,1333	0,091	Not OK
B	10905,369	0,000			1500	0	0,000	0,00	11,54	OK			

From Tables 5.24 and 5.25, it is known that there are stability ratio values greater than 0.1 on floors P1 to P3 for the x-direction earthquake, and on floors P1 and P1m for the y-direction earthquake.

In the structure of building model 2, there are no portal columns that are discontinuous from the roof to the pile cap, there is no void area exceeding 50% of the diaphragm area, and there are no bent portal columns, so the irregularities of the discontinuity of the vertical load-bearing vertical plane, the discontinuity of the diaphragm, and the irregularity due to the perpendicular shift of the plane are ignored.

5.3 Comparison Chart Summary

In this subsection, several comparative graphs will be presented for each model, including story shear comparison graphs, mass irregularity comparison graphs, story drift and drift ratio graphs, and P-Delta analysis. These graphs aim to examine the structural behavior differences resulting from variations in the beam axis.

5.3.1 Comparison Chart Story Shear

The following presents a comparison chart of story shear occurring at each floor level for all analyzed models. This chart provides insights into the variations in story shear across different structural models. The story shear comparison chart can be seen in Figure 5.3.

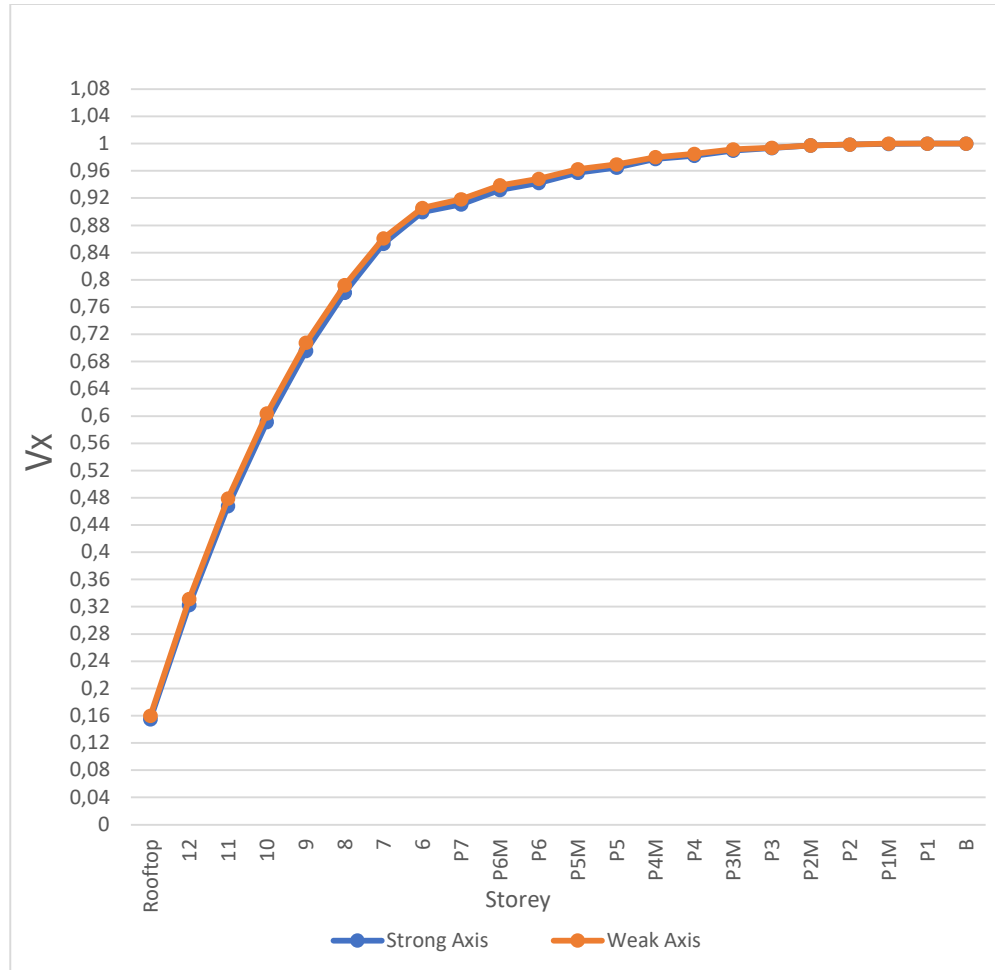


Figure 5. 3 Comparison Of Story Shear

From Figure 5.3, it can be observed that the V_x values, representing the total design lateral force, are higher in buildings utilizing weak-axis beams (Model 2) compared to those with strong-axis beams (Model 1). This may indicate a reduction in the structural stiffness of Model 2.

5.3.2 Comparison Chart of Stiffness Irregularity

The following presents a comparison chart of stiffness irregularity observed at each floor level for all analyzed models. This chart aims to provide a detailed representation of the stiffness distribution across floors. The comparison chart of stiffness irregularity in X direction can be seen in Figure 5.4 and the comparison chart of stiffness irregularity in Y direction can be seen in Figure 5.5.

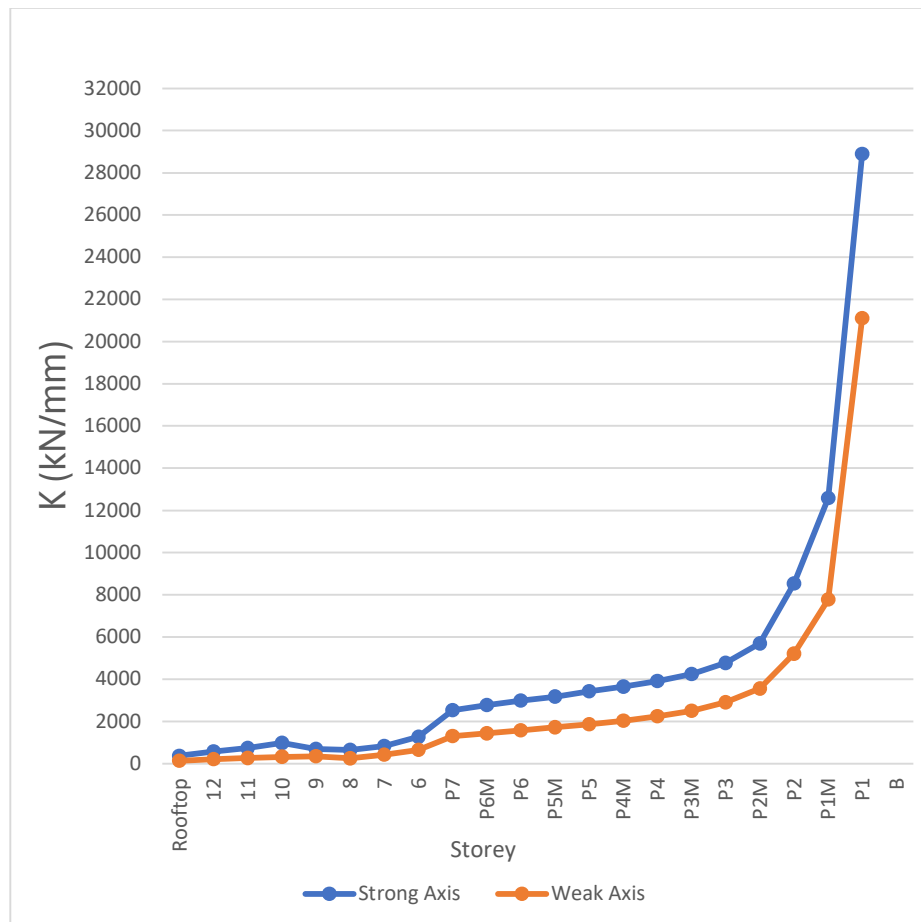


Figure 5. 4 Soft Story Stiffness at X direction

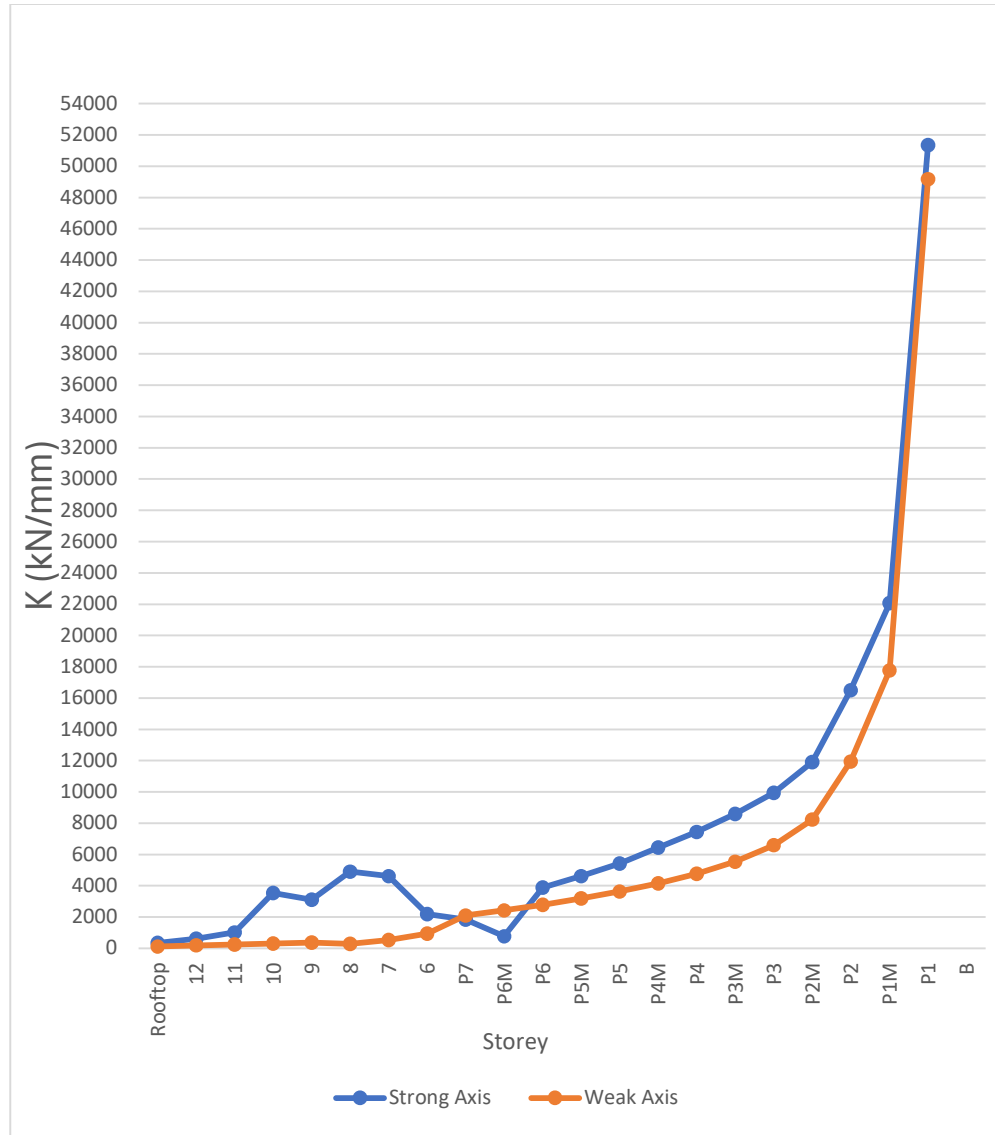


Figure 5. 5 Soft Story Stiffness at Y direction

When examining Figures 5.4 and 5.5, the K values representing structural stiffness in buildings utilizing weak-axis beams (Model 2) are lower compared to those with strong-axis beams (Model 1). This indicates that Model 2 experiences a reduction in structural stiffness. This conclusion is further supported by the V_x values, where the V_x of Model 2 is lower than that of Model 1, and by the increase in the period of Model 2 from 1.719 seconds to 1.9015 seconds.

5.3.3 Comparison Chart of Story Mass

The following presents a comparison chart of story mass observed at each floor level for all analyzed models. This chart is designed to provide a more comprehensive understanding of the mass distribution across floors. The comparison chart of story mass can be seen in Figure 5.6.

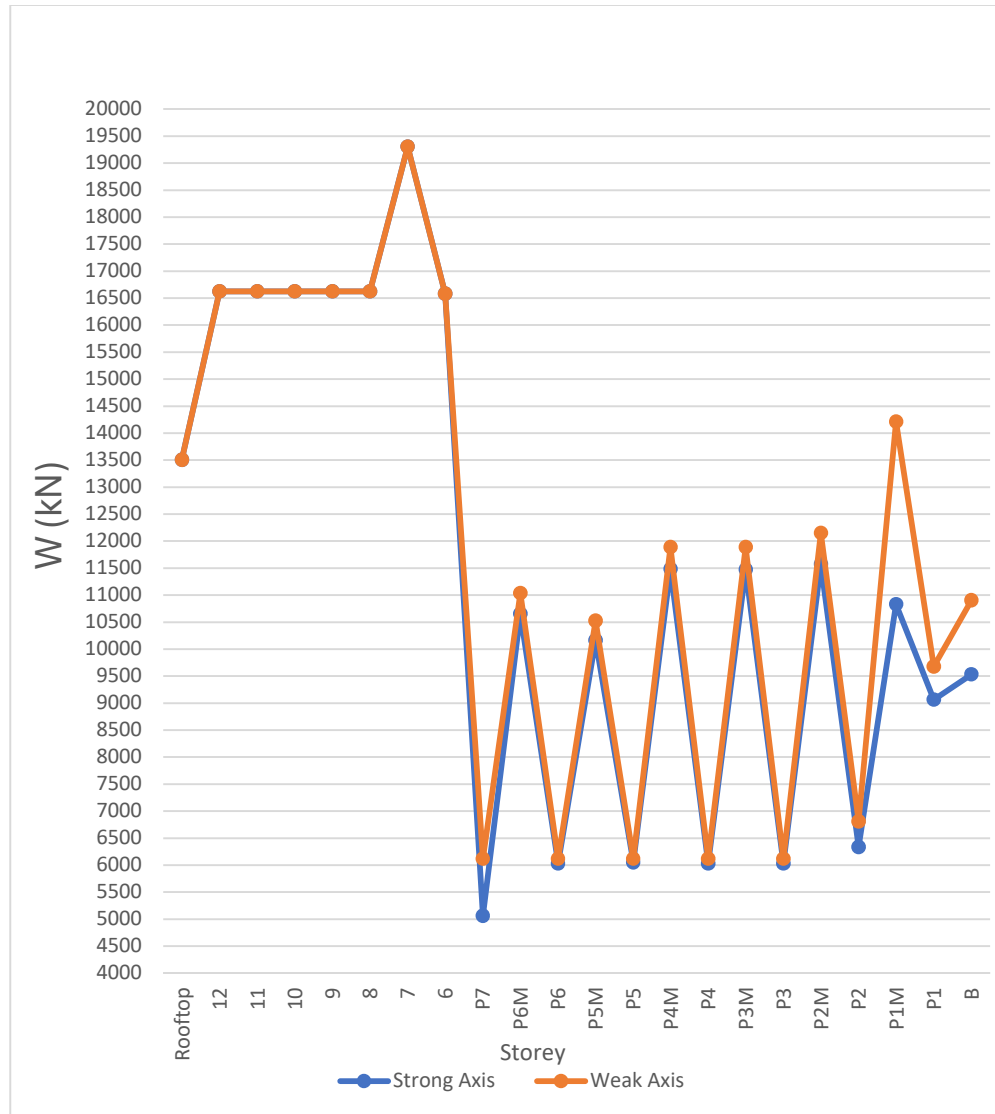


Figure 5. 6 Comparison of Story Mass

It can be seen from Figure 5.6 that the building utilizing weak-axis beams (Model 2) experiences an increase in mass from floor B to P7. This is caused by changes in beam dimensions, as exemplified by beam B1-a, which changed from 400 x 800 mm to 800 x 500 mm.

5.3.4 Comparison Chart of Story Drift and Drift Ratio

The following presents a comparison of the displacement graphs against the height of the building and the displacement ratio graphs between floors of the building in strong axis and weak axis modelling. The comparison graphs of the spectral response displacement on the x-axis and y-axis can be seen in Figures 5.3 and 5.4, and the comparison graphs of the displacement ratio between levels on the x-axis and y-axis can be seen in Figures 5.5 and 5.6.

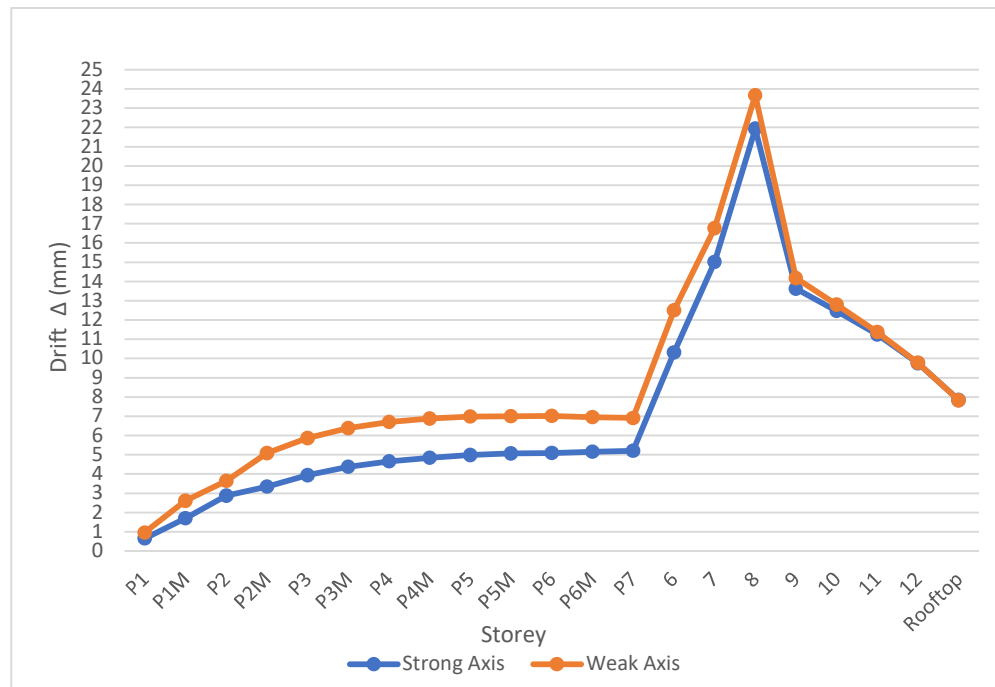


Figure 5. 7 Comparison of Story Drift X-Direction

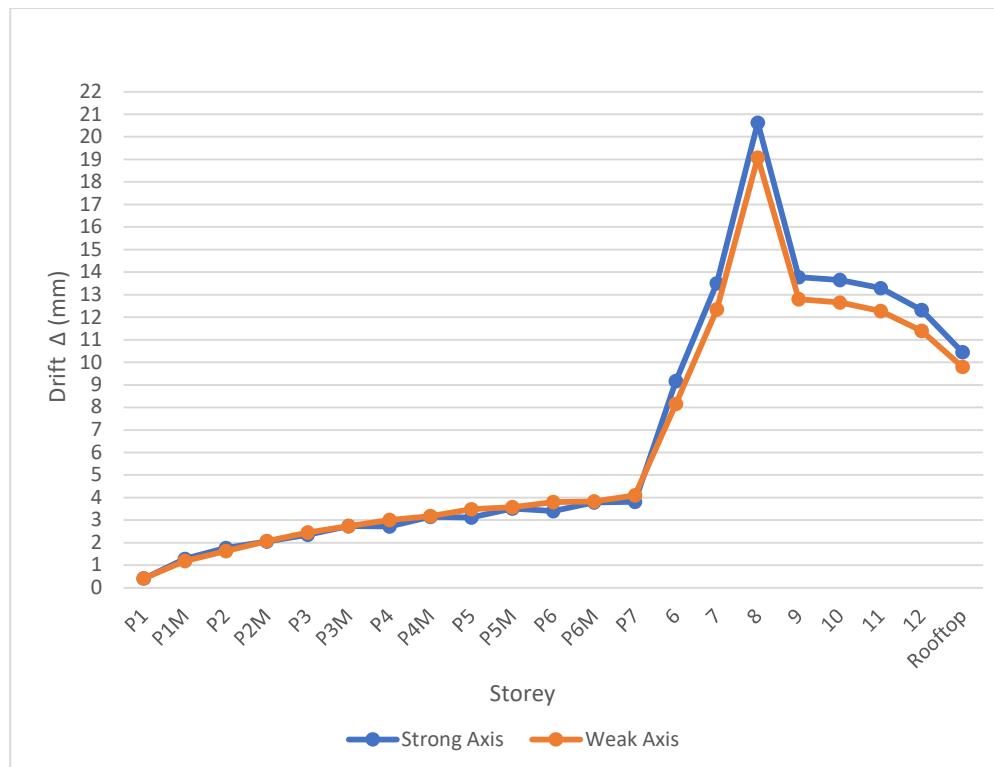


Figure 5. 8 Comparison of Story Drift Y-Direction

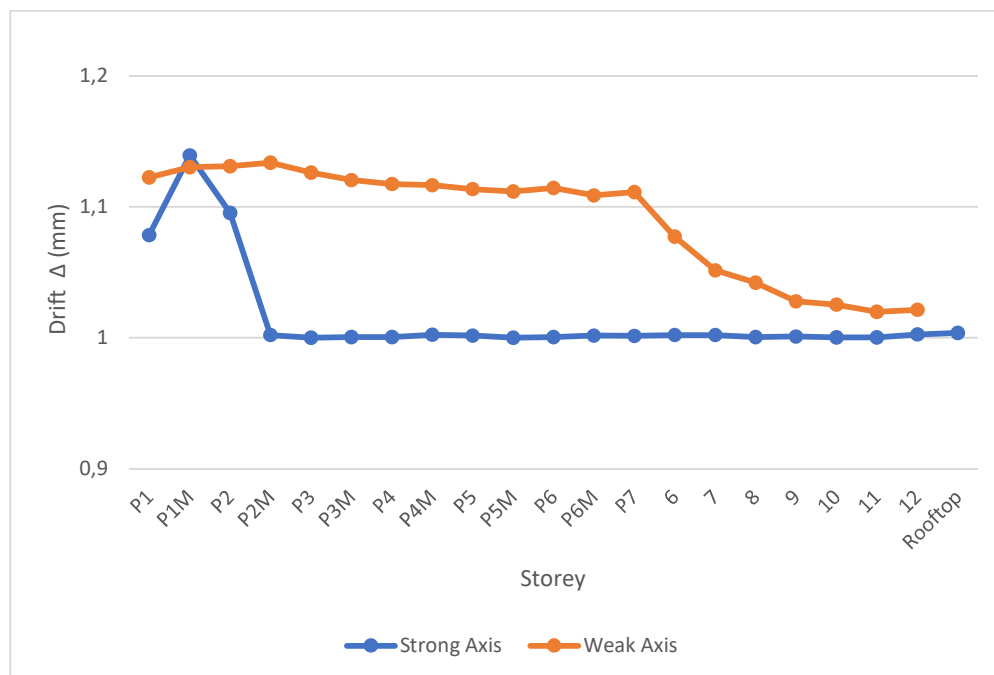


Figure 5. 9 Comparison of Story Drift Ratio X-Direction

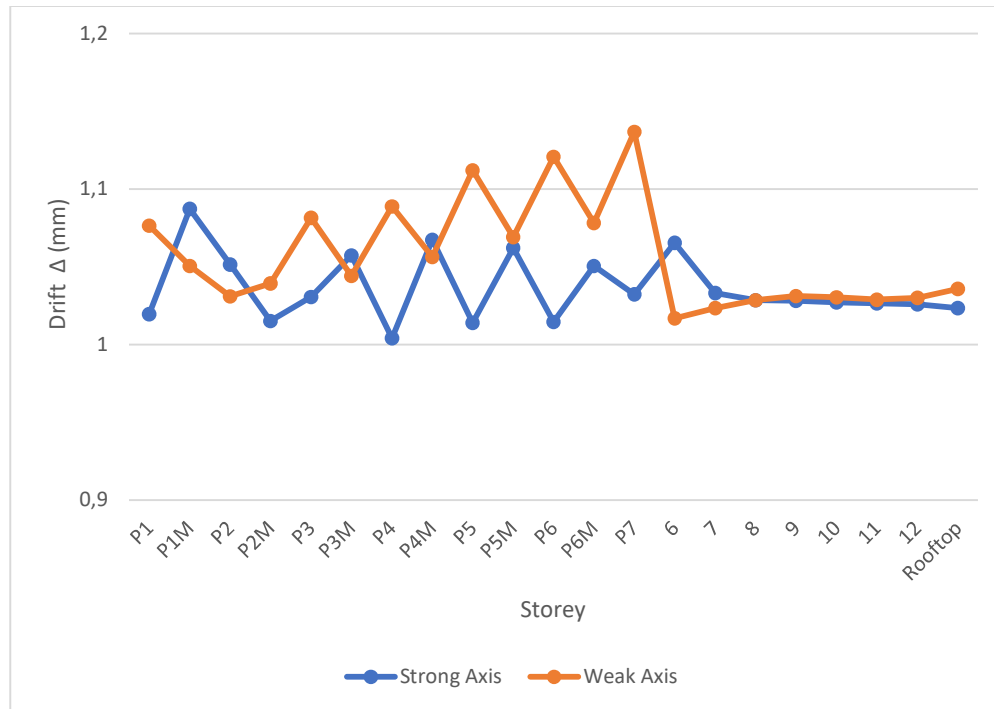


Figure 5. 10 Comparison of Story Drift Ratio Y-Direction

Based on the graph above, the deflection of the beam with the weak axis (Model 2) tends to be higher compared to the beam with the strong axis (Model 1), especially during an earthquake in the x direction. The change in beam orientation from the strong axis to the weak axis causes an uneven distribution of lateral loads, which in turn increases deformation or deflection in the more flexible beams. In other words, the lower stiffness of beams with a weak axis result in increased inter-story drift, which can impact the overall structural performance of the building, especially in resisting larger lateral earthquake forces.

5.3.5 Comparison Chart of P-Delta Stability

The following presents a comparison chart of *P-Delta* stability observed at each floor level for all analyzed models. The comparison chart of *P-Delta* stability in X direction can be seen in Figure 5.11 and the comparison chart of *P-Delta* stability in Y direction can be seen in Figure 5.12.

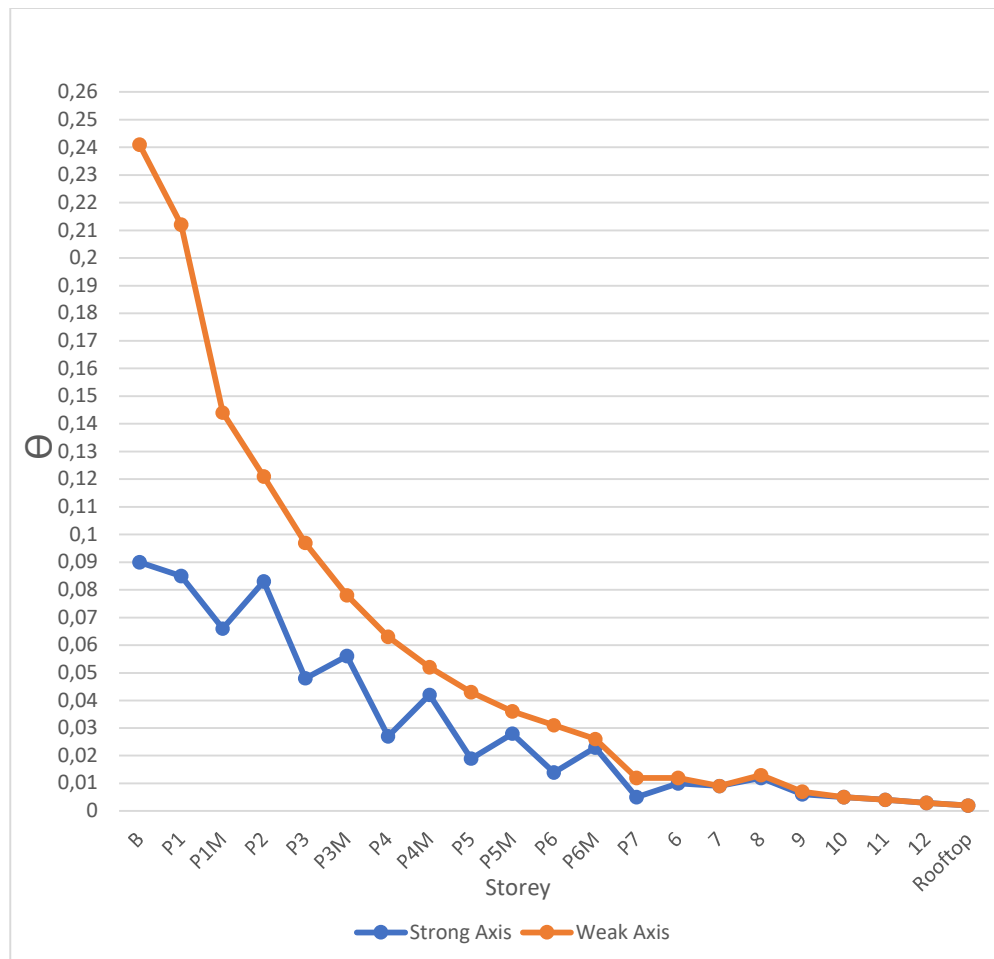


Figure 5. 11 Comparison chart of *P-Delta* stability in X direction

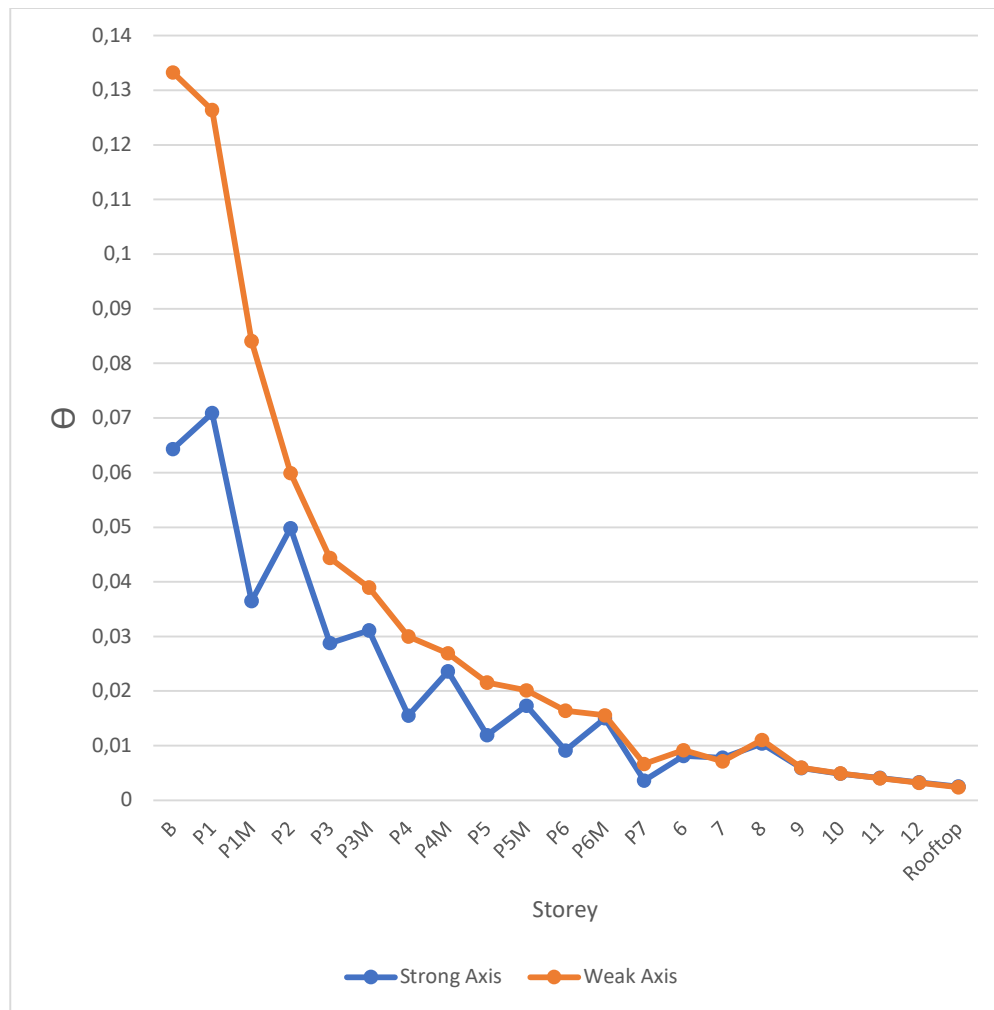


Figure 5. 12 Comparison chart of *P-Delta* stability in Y direction

From Figures 5.11 and 5.12, it is evident that the building utilizing weak-axis beams (Model 2) exhibits stability ratio values greater than 0.1 on floors P1 to P3 for earthquakes in the x-direction and on floors P1 and P1m for earthquakes in the y-direction. In contrast, the building with strong-axis beams (Model 1) has no stability ratio values exceeding 0.1. This indicates that Model 2 is deemed unsafe and requires redesign of its structural elements.

5.3.6 Comparison Chart of Beam Ultimate Moment

After conducting an analysis using modelling in the SAP 2000 application, a decrease in the ultimate moment of beams experiencing changes in the axis was found. The following are the comparison graphs of ultimate moments on strong axis beams and weak axis beams. The comparison graphs of ultimate moments for strong axis beams (model 1) and weak axis beams (model 2) in the support area can be seen in Figures 5.13 and 5.14. The comparison graphs of ultimate moments for strong axis beams (model 1) and weak axis beams (model 2) in the span area can be seen in Figures 5.15 and 5.16.

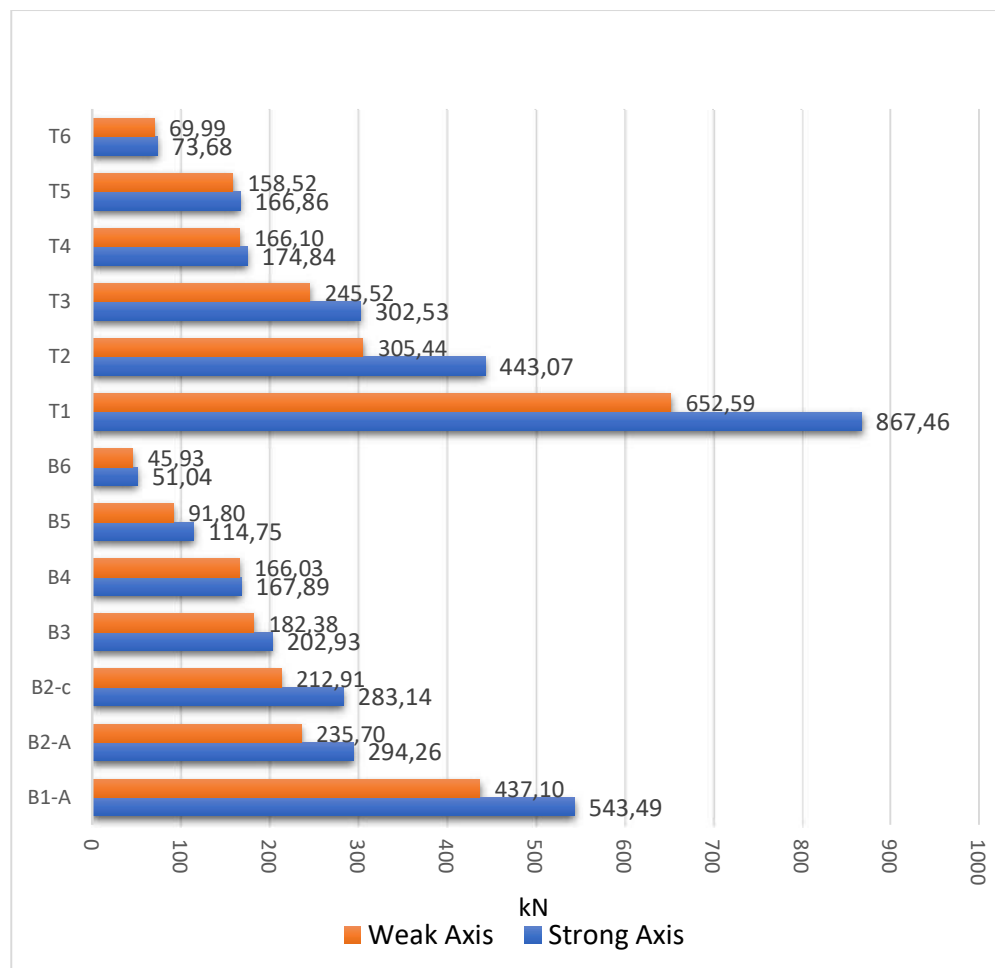


Figure 5. 13 Comparison of Mu+ at Support area

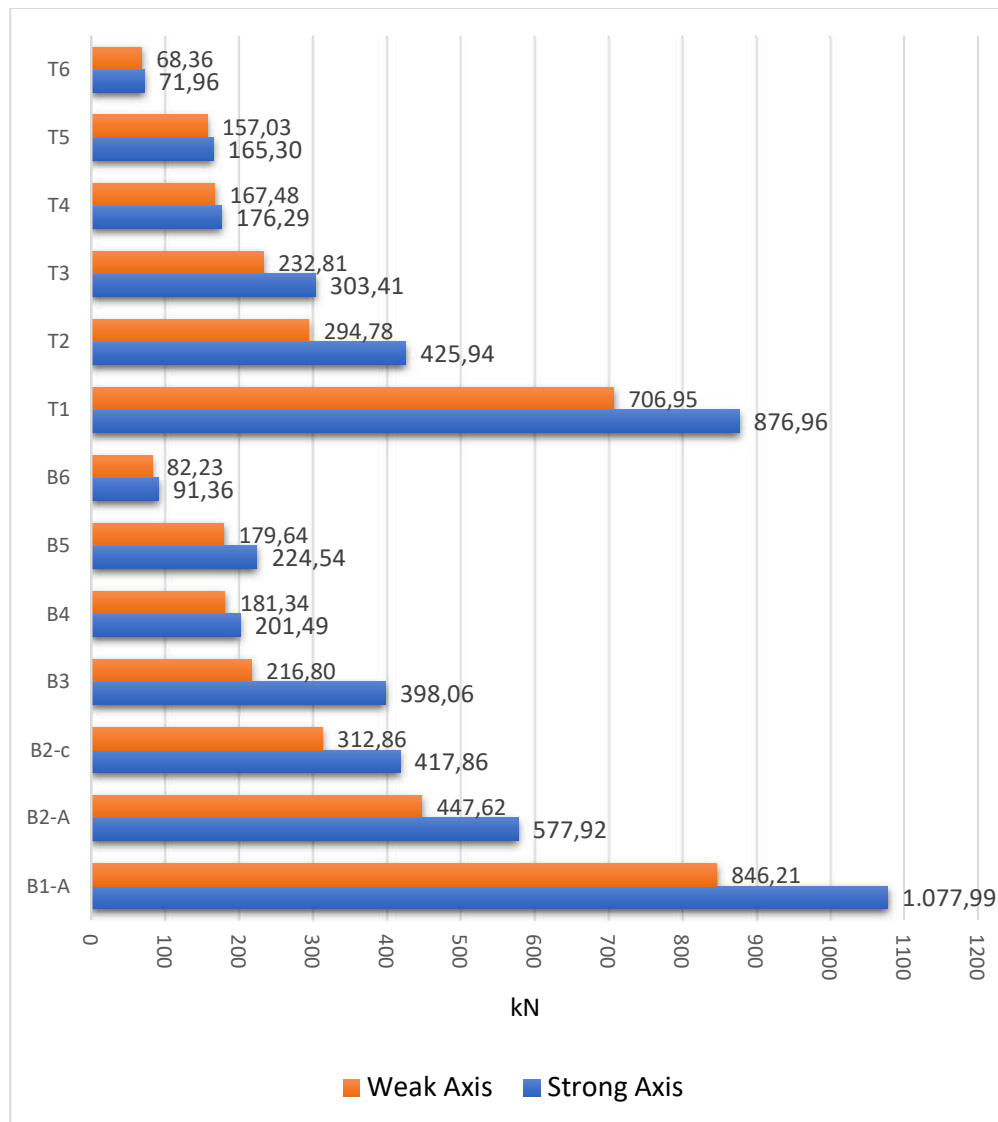


Figure 5. 14 Comparison of Mu- at Support area

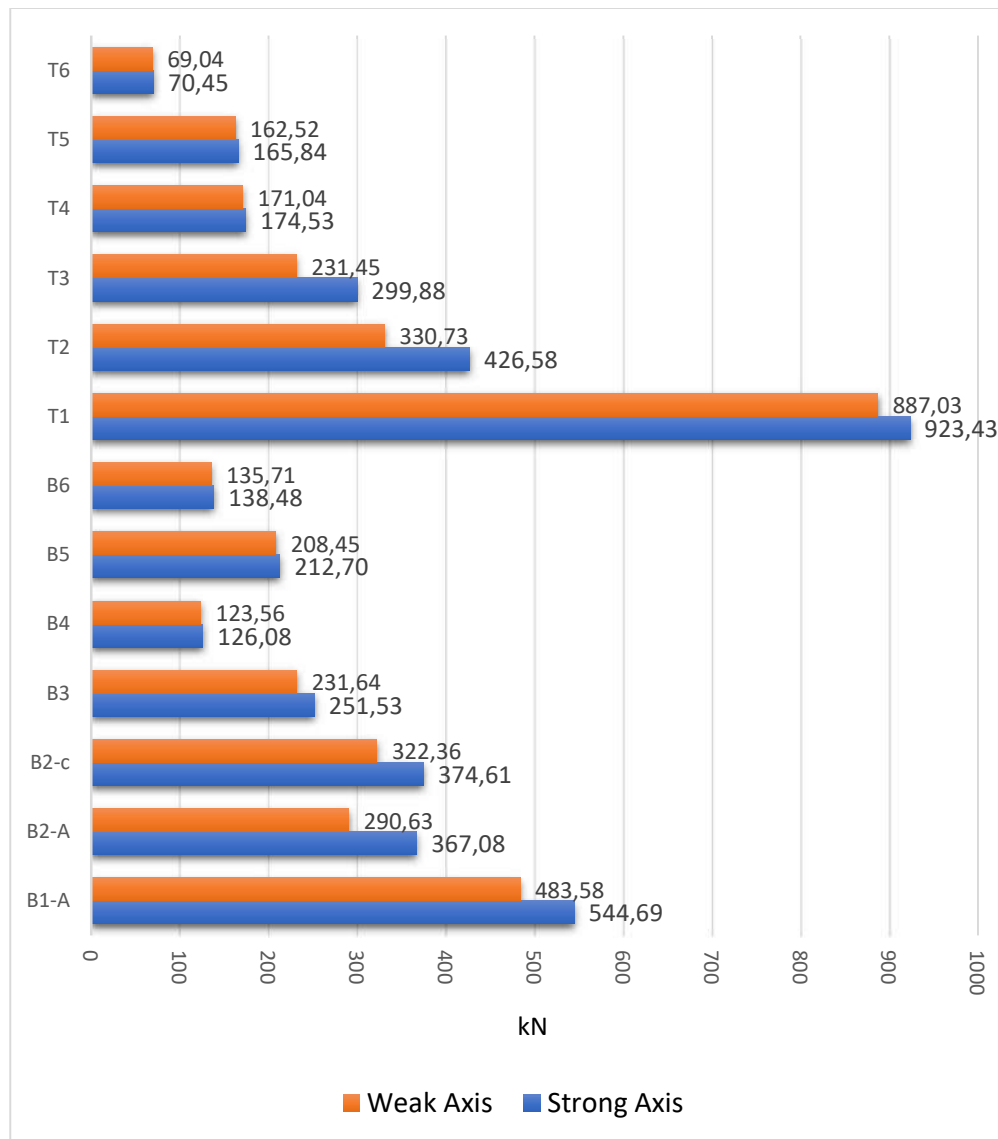


Figure 5. 15 Comparison of μ_u at Span area

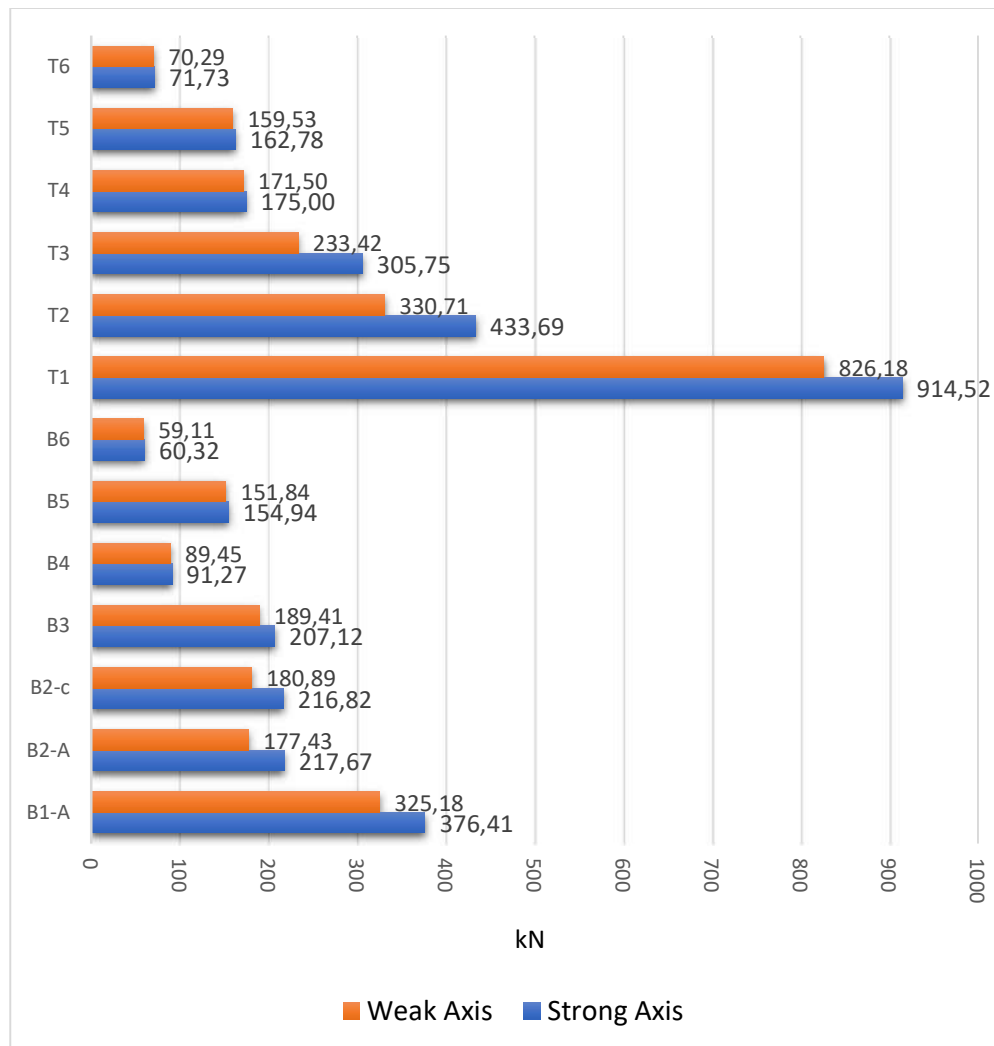


Figure 5. 16 Comparison of Mu- at Span area

From the comparison graph above, it can be clearly seen that there is a decrease in the ultimate moment values, both positive and negative, in the beam that undergoes a change in orientation from the strong axis to the weak axis. This decrease indicates that the change in the beam's axis orientation reduces the structural moment capacity that the beam can withstand. The average decrease in the moment value recorded was 10%, indicating a significant influence of the beam's orientation change on the overall strength and stability of the structure. This decrease in moment can affect the load distribution on the structure, so it needs to

be considered in the planning to ensure that the structural capacity of the building remains within safety standards, especially when facing large lateral loads.

5.4 Beam Reinforcement Design

5.4.1 Strong axis Beam ternal Forces and Reinforcement Design

The results of the internal force calculations for the strong axis beam model (model 1) were obtained from the initial design without any changes to the dimensions or the amount of reinforcement used in the beam. Presented in a table containing calculation parameters that include ultimate moment, nominal moment, diameter, and number of reinforcements. A summary of the reinforcement design calculations for the strong axis beam model (model 1) can be seen in table 5.24.

Table 5. 25 Beam Reinforcement Design Model 1

Beam	Cross-sectional Dimensions		Suport Area						Span Area						Shear Reinforcement	
	B	H	Top reinf	Bottom reinf	Mu-	Mu+	ϕ Mn-	ϕ Mn+	Bottom reinf	Top reinf	Mu-	Mu+	ϕ Mn-	ϕ Mn+	SP	LSP
B1-A	400	800	13D22	6D22	1.077,99	543,49	1265,16	601,573	6D22	4D22	376,41	544,69	406,563	601,256	4D10-150mm	2D10-200mm
B2-A	350	650	8D22	4D22	577,92	294,26	617,119	318,29	5D22	3D22	217,67	367,08	242,206	394,014	4D10-150mm	2D10-200mm
B2-c	350	650	6D22	4D22	417,86	283,14	454,735	317,663	5D22	3D22	216,82	374,61	242,206	394,014	2D10-150mm	2D10-200mm
B3	300	600	10D19	4D19	398,06	202,93	493,955	219,091	5D22	4D19	207,12	251,53	218,279	269,931	2D10-150mm	2D10-200mm
B4	300	500	6D19	4D22	201,49	167,89	246,602	175,544	3D19	3D19	91,27	126,08	134,267	134,267	2D10-150mm	2D10-200mm
B5	250	500	9D16	4D16	224,54	114,75	234,358	124,542	8D16	6D16	154,94	212,70	176,348	394,014	2D10-150mm	2D10-200mm
B6	200	400	5D16	2D16	365,45	433,80	502,995	559,784	8D16	4D16	150,80	692,40	639,9	834,591	2D10-100mm	2D10-50mm
T1	400	800	10D22	10D22	2.367,79	2.949,37	3239,95	3239,95	10D22	10D22	2.469,21	2.770,30	2998,14	2998,14	2D10-150mm	2D10-200mm
T2	350	650	6D22	6D22	1.192,64	1.240,60	1731,66	1731,66	6D22	6D22	1.344,45	1.365,05	1731,66	1731,66	2D10-150mm	2D10-200mm
T3	300	600	6D19	6D19	819,21	786,57	1150,33	1079,82	6D19	6D19	733,81	108,18	1150,23	1150,23	2D10-150mm	2D10-200mm
T4	300	500	4D19	4D19	634,65	629,44	669,256	669,256	4D19	4D19	629,98	645,76	669,256	669,256	2D10-150mm	2D10-200mm

Continued Table 5.24 Beam Reinforcement Design Model 1

Beam	Cross-sectional Dimensions		Support Area						Span Area						Shear Reinforcement	
	B	H	Top reinf	Bottom reinf	Mu-	Mu+	ϕ Mn-	ϕ Mn+	Bottom reinf	Top reinf	Mu-	Mu+	ϕ Mn-	ϕ Mn+	SP	LSP
T5	250	500	6D16	6D16	578,55	567,33	609,891	609,891	6D16	6D16	537,18	288,33	609,815	609,815	2D10-150mm	2D10-200mm
T6	200	400	3D16	3D16	251,87	169,46	304,693	301,912	3D16	3D16	229,53	246,58	301,912	301,912	2D10-150mm	2D10-200mm

5.4.2 Weak axis Beam Internal Forces and Reinforcement Design

Based on structural analysis using the SAP2000 application with a planned load combination. The data will be used as the basis for calculating reinforcement on the weak axis beam. (model 2). Here is an example of the nominal moment calculation for beam B1-A rotated in the support area,

1. Estimation of Beam Dimensions

Fy	= 420	MPa
Fc'	= 30	MPa
Mu+	= 437,099	kNm
Mu-	= 846,214	kNm
Mn	= Mu-/Ø	
	= 940,238 kNm	
εy	= fy/Es	
	= 0,0021	
m	= fy/0,85Fc'	
	= 16,47058824	
β1	= 0,85-(fc'-28/7 * 0,05)	
	= 0,836	
pb	= (β1/m) x (εc/(εc+εy))	
	= 0,029	
Rb	= pb x fy x (1-(0,5 x pb x m))	
	= 9,454 MPa	
Rm	= 0,75 x Rb	
	= 7,090 MPa	
H used	= 500	mm
B used	= 800	mm
Diameter of the main reinforcement	= 22	mm
Diameter of the shear reinforcement	= 10	mm
Concrete cover	= 40	mm
Vertical spacing of reinforcement	= 25	mm

Overstrength factor = 1,25

2. Single Reinforcement Component

Check μ_u requirements

μ_u > 50% μ_u

$C_{used} = 0,2$

$R_1 = c \times R_b$
 $= 1,890 \text{ Mpa}$

$M_1 = R_1 \times b \times h_p \times h_p^2$
 $= 1,890 \times 800 \times 500^2 = 378178271,3 \text{ Nmm}$

$d_{s_{assume}} = P_b + D_{shear} + D_{main} + 0,5 \times 25$
 $= 84,5 \text{ mm}$

$d = h - d_s$
 $= 415,5 \text{ mm}$

$a = a_{use}$
 $= 47,309 \text{ mm}$

$c = a / \beta_1$
 $= 56,610 \text{ mm}$

$\epsilon_s = ((d - c) / c) \times 0,003$
 $= 0,019$

$\epsilon_s > \epsilon_y$

OK, the steel has yield.

$F_y = 420 \text{ Mpa}$

$T_s = CC$

$T_s = 0,85 F_c' a b$

$A_{s_{need}} \times F_y = 0,85 F_c' a b$

$A_{s_{need}} = 0,85 F_c' a b / f_y$
 $= 2297,909 \text{ mm}^2$

$A_1 \text{ Reinf} = 1/4 \pi d_{main}^2$
 $= 380,132 \text{ mm}^2$

$$\begin{aligned}\text{The amount of reinforcement needed} &= A_s \text{ need} / A_1 \text{ Reinf} \\ &= 6,045019062 \text{ pieces}\end{aligned}$$

The number of rebars should be rounded up to 8 pieces.

$$\begin{aligned}A_s \text{ new} &= \text{Number of reinforcement bars used} \times A_1 \text{ Reinf} \\ &= 3041,061689 \text{ mm}^2\end{aligned}$$

$$\begin{aligned}T_s \text{ new} &= A_s \text{ new} \times \text{teg baja pakai} \\ &= 1277245,909 \text{ N}\end{aligned}$$

$$T_s \text{ new} = 0,85 f_c' a b$$

$$\begin{aligned}a \text{ new} &= T_s \text{ new} / 0,85 f_c' b \\ &= 62,61009359 \text{ mm}\end{aligned}$$

$$\begin{aligned}c \text{ new} &= a \text{ new} / \beta_1 \\ &= 74,91806071 \text{ mm}\end{aligned}$$

$$\epsilon_s \text{ new} = ((d - c \text{ new}) / c \text{ new}) \times 0.003 = 0,013638178$$

$$\epsilon_s \text{ new} > \epsilon_y$$

OK, the steel has yield.

$$\begin{aligned}M_n1 &= C_c \times (d - 0,5a) \\ &= 490711432,3 \text{ Nmm}\end{aligned}$$

3. Composite Reinforcement Components

$$\begin{aligned}M_n2 &= M_n - M_n1 \\ &= 449527345,4 \text{ Nmm}\end{aligned}$$

$$\begin{aligned}d_s' \text{ asumsi} &= P_b + D_{seng} + 0,5D_{pok} \\ &= 61 \text{ mm}\end{aligned}$$

$$\begin{aligned}d &= h - d_s \\ &= 415,5 \text{ mm}\end{aligned}$$

$$M_n2 = C_s \times (d - d_s')$$

$$A_s' \times F_y = M_n2 / (d - d_s')$$

$$\begin{aligned}A_s' \text{ need} &= (M_n2 / (d - d_s')) / F_y \\ &= 3019,190983 \text{ mm}^2\end{aligned}$$

$$\begin{aligned}A_1 \text{ reinf} &= 1/4 \phi d^2 \\ &= 380,1327111 \text{ mm}^2\end{aligned}$$

num reinforcement

$$\begin{aligned} \text{need} &= A_s' \text{ need} / A_1 \text{ reinf} \\ &= 7,942465604 \text{ pieces} \end{aligned}$$

Number of reinforcement bars, round up to 8 pieces.

4. Negative Moment Available

Number of Reinforcements Used

$$\begin{aligned} \text{Upper (tension)} &= \text{single reinf} + \text{composite reinf} \\ &= 16 \text{ pieces} \\ \text{Lower (compression)} &= \text{composite reinf} \\ &= 8 \text{ pieces} \end{aligned}$$

Tensile reinforcement configuration

Centre of Gravity of Reinforcement

$$\text{First line} = 12 \text{ pieces}$$

$$\text{Second line} = 4 \text{ pieces}$$

$$\begin{aligned} \text{Line spacing} &= 25 + 1D \text{ main reinf} \\ &= 51 \text{ mm} \end{aligned}$$

$$\begin{aligned} x_1 &= P_b + D_{\text{shear}} + 0,5D_{\text{pok}} \\ &= 61 \text{ mm} \end{aligned}$$

$$\begin{aligned} x_2 &= X_1 + \text{Line spacing} \\ &= 112 \text{ mm} \end{aligned}$$

$$\begin{aligned} x_3 &= X_2 + \text{Line spacing} \\ &= 163 \text{ mm} \end{aligned}$$

$$\begin{aligned} A_{s1} &= \mu_{\text{un}} \text{ First line } 1 \times A_1 \text{ reinf} \\ &= 4561,592533 \text{ mm}^2 \end{aligned}$$

$$\begin{aligned} A_{s2} &= \mu_{\text{un}} \text{ First line } 2 \times A_1 \text{ reinf} \\ &= 1520,530 \text{ mm}^2 \end{aligned}$$

$$A_{s1} \cdot X_1 = 278257,144 \text{ mm}^3$$

$$\begin{aligned} A_{s2} \cdot x_2 &= 170299,454 \text{ mm}^3 \text{ ds} = (\Sigma A_s \cdot X) / (\Sigma A_s) \\ &73,750 \text{ mm} \end{aligned}$$

Check the distance between horizontal reinforcements

$$S = (B_{\text{use}} - (2 \cdot p_b) - (2 \cdot \text{shear diameter}) - (\text{num of reinf} \cdot D_{\text{reinf}})) / (\text{num of reinf} - 1)$$

$$= 39,636 \text{ mm}$$

$$S > 25 \quad \text{OK}$$

Compression reinforcement configuration

$$\begin{aligned} \text{First line} &= 8 \text{ pircs} \\ \text{Line spacing} &= 25 + 1D_{\text{main reinf}} \\ &= 51 \text{ mm} \\ x_1 &= p_b + D_{\text{shear}} + 0,5D_{\text{main}} \\ &= 61 \text{ mm} \\ x_2 &= x_1 + \text{Line spacing} \\ &= 112 \text{ mm} \\ x_3 &= x_2 + \text{Line spacing} \\ &= 163 \text{ mm} \\ A_{s1} &= \text{num First line} \times A_1 \text{ reinf} \\ &= 3041,061689 \text{ mm}^2 \\ A_{s1} \cdot x_1 &= 185504,763 \text{ mm}^3 \text{ ds}' = (\Sigma A_s \cdot X) / (\Sigma a_s) \\ &= 61,000 \text{ mm} \end{aligned}$$

Check the distance between horizontal reinforcements.

$$S = (B_{\text{use}} - (2 \cdot p_b) - (2 \cdot \text{shear diameter}) - (\text{mun of reinf} \cdot D_{\text{reinf}})) / (\text{mun of reinf} - 1)$$

$$= 74,857 \text{ mm}$$

$$S > 25 \quad \text{OK}$$

5. Calculation of Moment Nominal -

$$\begin{aligned} d_s &= 73,750 \text{ mm} \\ d_s' &= 61,0 \text{ mm} \\ d &= h - d_s = 426,250 \text{ mm} \\ A_{s \text{ use}} &= \text{Num of tensile reinforcements used} \times A_1 \text{ reinf} \\ &= 6082,123377 \text{ mm}^2 \end{aligned}$$

$$\begin{aligned}
A_s' \text{ use} &= \text{Num of compression reinforcement bars used} \times A_{1 \text{ reinf}} \\
&= 3041,061689 \text{ mm}^2 \\
T_s &= C_c + C_s \\
A_s \times F_y &= (0,85F_c' \times c \times B_1 \times b_{\text{use}}) + (A_s' \times (\epsilon_c(c-d_s')/c) \times E_s) \\
\\
C &= 104,992 \text{ mm} \\
a &= C \times B_1 \\
&= 87,743 \text{ mm} \\
F_s &= (0,003 \times (c-d_s')/c) \times E_s \\
&= 251,401 \text{ Mpa} \\
M_n &= C_c \times (d-0,5a) + C_s \times (d-d_s') \\
C_c &= 0,85F_c' \times a \times b \\
&= 1789963,097 \text{ N} \\
C_s &= A_s' \times F_s \\
&= 764528,7213 \text{ N} \\
\text{Firs arm} &= d-0,5a \quad 382,378 \quad \text{mm} \\
\text{Second arm} &= d-d_s' \quad 365,250 \quad \text{mm} \\
M_n &= C_c \times (d-0,5a) + C_s \times (d-d_s') \quad 963687260,9 \text{ Nmm} \\
\epsilon_s &= 0,009179503 \\
\epsilon_s' &= 0,00125701 \\
\epsilon_t &= ((D-c)/c)-0.003 \\
&= 0,008872337 \\
\phi &= 0,9 \\
\phi M_n &= M_n \times \phi \\
&= 867318534,8 \text{ Nmm} \\
&= 867,3185348 \text{ kNm} \\
\\
867,3185348 &> 846,2149 \text{ kNm} \\
\phi M_n &> \quad \quad \quad \mu_u \quad \quad \quad \text{OK}
\end{aligned}$$

6. Check Nominal Moment Positive

$$\begin{aligned}
 d_s &= 61,0 \text{ mm} \\
 d_s' &= 73,750 \text{ mm} \\
 d &= h - d_s \\
 &= 439,0 \text{ mm} \\
 A_s \text{ used} &= \text{Num of tensile reinforcements used} \times A_{1 \text{ tul}} \\
 &= 3041,062 \text{ mm}^2 \\
 A_s' \text{ used} &= \text{Num of compression reinforcement bars used} \times A_{1 \text{ reinf}} \\
 &= 6082,123 \text{ mm}^2 \\
 T_s &= C_c + C_s \\
 A_s \times F_y &= (0,85F_c' \times c \times B_1 \times b \text{ use}) + (A_s' \times (\epsilon_c(c-d_s')/c) \times E_s \\
 \\
 C &= 74,050 \text{ mm} \\
 a &= c \times B_1 \text{ 61,88483027 mm} \\
 F_s &= (0,003 \times (c-d_s')/c) \times E_s \\
 &= 2,432599727 \text{ Mpa} \\
 M_n &= C_c \times (d-0,5a) + C_s \times (d-d_s') \\
 C_c &= 0,85F_c' \times a \times b \\
 &= 1262450,538 \text{ N} \\
 C_s &= A_s' \times F_s = 14795,371 \text{ N} \\
 d-0,5a &= 408,1 \text{ mm} \\
 d-d_s' &= 365,250 \text{ mm} \\
 M_n &= C_c \times (d-0,5a) + C_s \times (d-d_s') \\
 &= 520556526,9 \text{ Nmm} \\
 \epsilon_s &= 0,014785226 \\
 \epsilon_s' &= 1,2163E-05 \\
 \epsilon_t &= ((D-c)/c) - 0.003 = 0,01383317 \\
 \phi &= 0,9
 \end{aligned}$$

$$\begin{aligned}
 \phi M_n &= M_n \times \phi = 468500874,2 \text{ Nmm} \\
 \phi M_n &= M_n \times \phi = 468,500 \text{ kNm} \\
 468,500 &> 437,09956 \text{ kNm} \\
 \phi M_n &> M_u+ \quad \text{OK aman}
 \end{aligned}$$

So that the flexural reinforcement B1A (Support Area):

$$\text{Top} = 16 \text{ D } 22$$

$$\text{Lower} = 8 \text{ D } 22$$

cross-section image of beam B1A weak axis at the support area can be seen on figure 5.11.

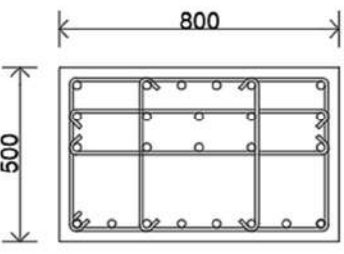
Type	B1-a Weak
Area	Support
Section	
CC	40 mm
Top Reinf	16D22
Bottom Reinf	8D22
Shear Reinf	4D10-150mm

Figure 5. 17 Beam Cross Section plan, B1A

The results of the reinforcement calculations for the cross-section of beam model 2 are presented in a table containing calculation parameters that include the ultimate moment, moment nominal, diameter and number of reinforcements, A summary of the internal forces and reinforcement design calculations for weak axis beam (model 2) can be seen in table 5.27, table 5.28, and table 5.29

Table 5. 26 Beam Reinforcement Design Model 2 Support Area

Beam	Cross-sectional Dimensions		Support Area							
	B	H	Top reinf	Bottom reinf	Mu-	Mu+	ϕM_n -	ϕM_n +	Cek ϕM_n -	Cek ϕM_n +
B1-A	800	500	16D22	8D22	846,21	437,10	867,319	468,501	OK	OK
B2-A	650	500	9D22	5D22	447,62	235,70	516,13	298,453	OK	OK
B2-c	650	500	6D22	4D22	312,86	212,91	353,315	243,444	OK	OK
B3	600	500	7D19	4D19	216,80	182,38	309,85	185,856	OK	OK
B4	300	500	6D19	5D22	181,34	166,03	246,602	175,544	OK	OK
B5	250	500	9D16	4D16	179,64	91,80	234,358	124,542	OK	OK
B6	200	400	5D16	2D16	82,23	45,93	108,534	52,5766	OK	OK
T1	850	650	12D22	12D22	706,95	652,59	946,654	946,654	OK	OK
T2	650	500	7D22	7D22	294,78	305,44	407,845	407,845	OK	OK
T3	600	500	7D19	7D19	232,81	245,52	309,78	309,78	OK	OK
T4	300	500	4D19	4D19	167,48	166,10	180,34	180,34	OK	OK
T5	250	500	6D16	6D16	157,03	158,52	179,228	179,228	OK	OK
T6	200	400	3D16	3D16	68,36	69,99	74,5597	74,5597	OK	OK

Table 5. 27 Beam Reinforcement Design Model 2 Span Area

Beam	Cross-sectional Dimensions		Span Area							
	B	Bottom reinf	Top reinf	Mu-	Mu+	ϕ Mn-	ϕ Mn+	Cek ϕ Mn-	Cek ϕ Mn+	Cek ϕ Mn+
B1-A	800	9D22	6D22	325,18	483,58	358,865	522,986	OK	OK	OK
B2-A	650	6D22	4D22	177,43	290,63	243,443	353,314	OK	OK	OK
B2-c	650	6D22	3D22	180,89	322,36	188,381	353,362	OK	OK	OK
B3	600	6D22	5D19	189,41	231,64	227,162	268,513	OK	OK	OK
B4	300	3D19	3D19	89,45	123,56	134,266	134,266	OK	OK	OK
B5	250	8D16	6D16	151,84	208,45	176,347	353,362	OK	OK	OK
B6	200	8D16	4D16	59,11	135,71	88,335	142,321	OK	OK	OK
T1	850	12D22	12D22	826,18	887,03	946,653	946,653	OK	OK	OK
T2	650	7D22	7D22	330,71	330,73	407,844	407,844	OK	OK	OK
T3	600	7D19	7D19	233,42	231,45	309,780	309,780	OK	OK	OK
T4	300	4D19	4D19	171,50	171,04	180,340	180,340	OK	OK	OK
T5	250	6D16	6D16	159,53	162,52	182,858	182,858	OK	OK	OK
T6	200	3D16	3D16	70,29	69,04	74,5597	74,559	OK	OK	OK

Table 5. 28 Beam Shear Reinforcement Design Model 2

Beam	Cross-sectional Dimensions		Shear Reinforcement	
	B	H	SP	LSP
B1-A	800	500	4D10-150mm	2D10-200mm
B2-A	650	500	4D10-150mm	3D10-170mm
B2-c	650	500	3D10-150mm	2D10-200mm
B3	600	500	2D10-150mm	2D10-200mm
B4	300	500	2D10-150mm	2D10-200mm
B5	250	500	2D10-150mm	2D10-200mm
B6	200	400	2D10-100mm	2D10-50mm
T1	850	650	3D10-130mm	4D10-200mm
T2	650	500	3D10-150mm	2D10-200mm
T3	600	500	2D10-150mm	2D10-200mm
T4	300	500	2D10-150mm	2D10-200mm
T5	250	500	2D10-150mm	2D10-200mm
T6	200	400	2D10-150mm	2D10-200mm

5.4.2 Comparison table of Beam Reinforcement Area

After conducting an internal forces analysis on the weak axis beam and redesigning the reinforcement requirements for the weak axis beam, it turns out that the reinforcement requirements for the beam increased even though the ultimate moment on that beam is smaller compared to the strong axis beam. The increase can be seen in table 5.27 below.

Table 5. 29 Comparison of Beam Reinforcement Area

Beam	Area of Flexural Reinforcement (mm ²)						Shear Reinforcement			
	Support Area			Span Area			Plastic Hinge		Outside Plastic Hinge	
	Strong Axis	Weak Axis	% Increase	Strong Axis	Weak Axis	% Increase	Strong Axis	Weak Axis	Strong Axis	Weak Axis
B1-A	7222,521	9123,185	26,315	3801,327	5701,990	50	4D10-150mm	4D10-150mm	2D10-200mm	2D10-200mm
B2-A	4561,592	5321,857	16,666	3041,061	3801,327	25	4D10-150mm	4D10-150mm	2D10-200mm	3D10-170mm
B2-c	3801,327	3801,327	0	3041,061	3421,194	12,5	2D10-150mm	3D10-150mm	2D10-200mm	2D10-200mm
B3	3969,402	3118,816	-21,428	2551,758	3118,816	22,222	2D10-150mm	2D10-150mm	2D10-200mm	2D10-200mm
B4	2835,287	2835,287	0	1701,172	1701,172	0	2D10-150mm	2D10-150mm	2D10-200mm	2D10-200mm
B5	2613,805	2613,805	0	2814,867	2814,867	0	2D10-150mm	2D10-150mm	2D10-200mm	2D10-200mm
B6	1407,433	1407,433	0	2412,7431	2412,743	0	2D10-100mm	2D10-100mm	2D10-150mm	2D10-150mm
T1	7602,654	9123,185	20	7602,654	9123,185	20	2D10-150mm	3D10-130mm	2D10-200mm	4D10-200mm
T2	4561,592	5321,857	16,666	4561,592	5321,857	16,666	2D10-150mm	3D10-150mm	2D10-200mm	2D10-200mm
T3	3402,344	3969,402	16,666	3402,344	3969,402	16,666	2D10-150mm	2D10-150mm	2D10-200mm	2D10-200mm
T4	2268,229	2268,229	0	2268,229	2268,229	0	2D10-150mm	2D10-150mm	2D10-200mm	2D10-200mm
T5	2412,743	2412,743	0	2412,7431	2412,743	0	2D10-150mm	2D10-150mm	2D10-200mm	2D10-200mm
T6	1206,371	1206,371	0	1206,3715	1206,371	0	2D10-150mm	2D10-150mm	2D10-200mm	2D10-200mm
Average			12,481	Average		23,293				

From Table 5.29, although the ultimate moment in the weak axis beam decreases by about 10%, the reinforcement requirement increases by an average of 12.481% in the support area and 23.293% in the span area. This increase is due to the decrease in beam height, which reduces its capacity to resist pressure and bending moments. As a result, the overall stiffness decreases, requiring more reinforcement to ensure the beam can safely carry the applied load.

CHAPTER VI

CONCLUSION AND SUGGESTIONS

6.1 Conclusion

Based on the analysis results of the building that has a change in the main beam axis from the original strong axis beam to the weak axis beam, the following conclusion can be drawn:

1. In the building that uses a weak axis beam (model 2), there is an increase in the total building weight (W) compared to the building with a strong axis beam (model 1), from initially 255410.340 kN to 260174.052 kN. This can occur due to an increase in the dimensions of the main beam in the parking area, as exemplified by beam B1-a, which changed from 400 x 800 mm to 800 x 500 mm.
2. In buildings that use weak-axis beams (Model 2), the K values, which represent structural stiffness, are lower compared to those with strong-axis beams (Model 1). This indicates that Model 2 experiences a reduction in structural stiffness. This conclusion is further supported by the V_x values, where the V_x of Model 2 is lower than that of Model 1, as well as by the increase in the period of Model 2 from 1.719 seconds to 1.9015 seconds. Additionally, the V_x value or design lateral force in Model 2 tends to be higher compared to Model 1.
3. After conducting a comparative analysis of irregularities and P-delta effects, it was found that buildings utilizing weak-axis beams (Model 2) experience a decline in structural behavior. This conclusion is supported by several findings, including an increase in the building's fundamental period from 1.719 seconds to 1.9015 seconds, higher story shear (V_x) values on each floor, lower stiffness (K) values, higher story drift on each floor, and the presence of P-delta values exceeding 0.1 on floors P1-P3. This indicates that the structure of building model 2 is unstable and does not meet the

requirements of SNI 1726:2019, so a redesign of the structural elements is needed to comply with SNI 1726:2019.

4. After conducting redesigning the reinforcement requirements for the weak axis beam, it turns out that the reinforcement requirements for the beam increased even though the ultimate moment on that beam is smaller compared to the strong axis beam. The increase in reinforcement requirements for the beam is an average of 12.481% in the support area and 23.293% in the span area. This is attributed to the reduction in beam height (H), which is one of the primary factors influencing a beam's ability to resist forces.

6.2 Suggestions

Based on the research findings, several recommendations can be made for future studies, including the following.

1. Acquiring additional scholarly sources to enhance understanding of the fundamental principles of building structure planning.
2. In every structural planning, it is advisable to pay attention to the continuously evolving regulations. Because with the development of these regulations, there are bound to be changes from the previous ones. So that the planned building is safe and comfortable to function as intended.
3. A further comparative analysis is necessary using different building structures in terms of shape, dimensions, number of floors, floor-to-floor height, and material.
4. Future research can employ more efficient calculation methods and incorporate the latest requirements to conduct a similar comparative analysis.

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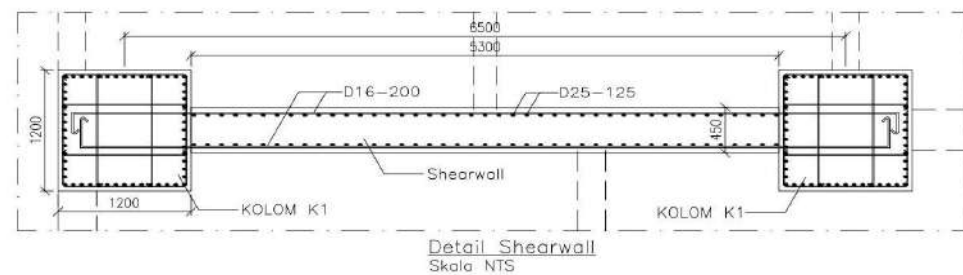
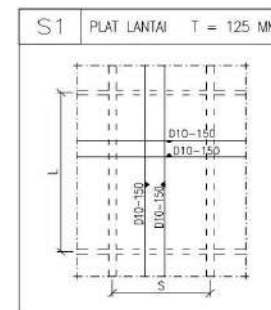
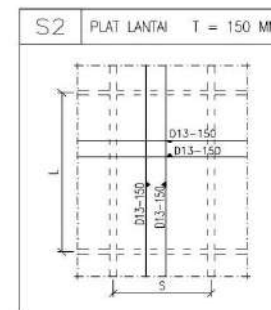
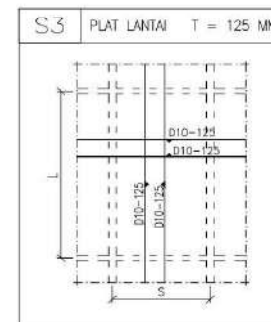
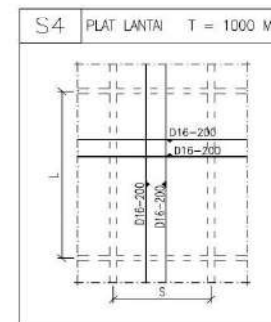
APPENDIX

Appendix 1 Detail engineering for beam, column, and slabs

[illegible]

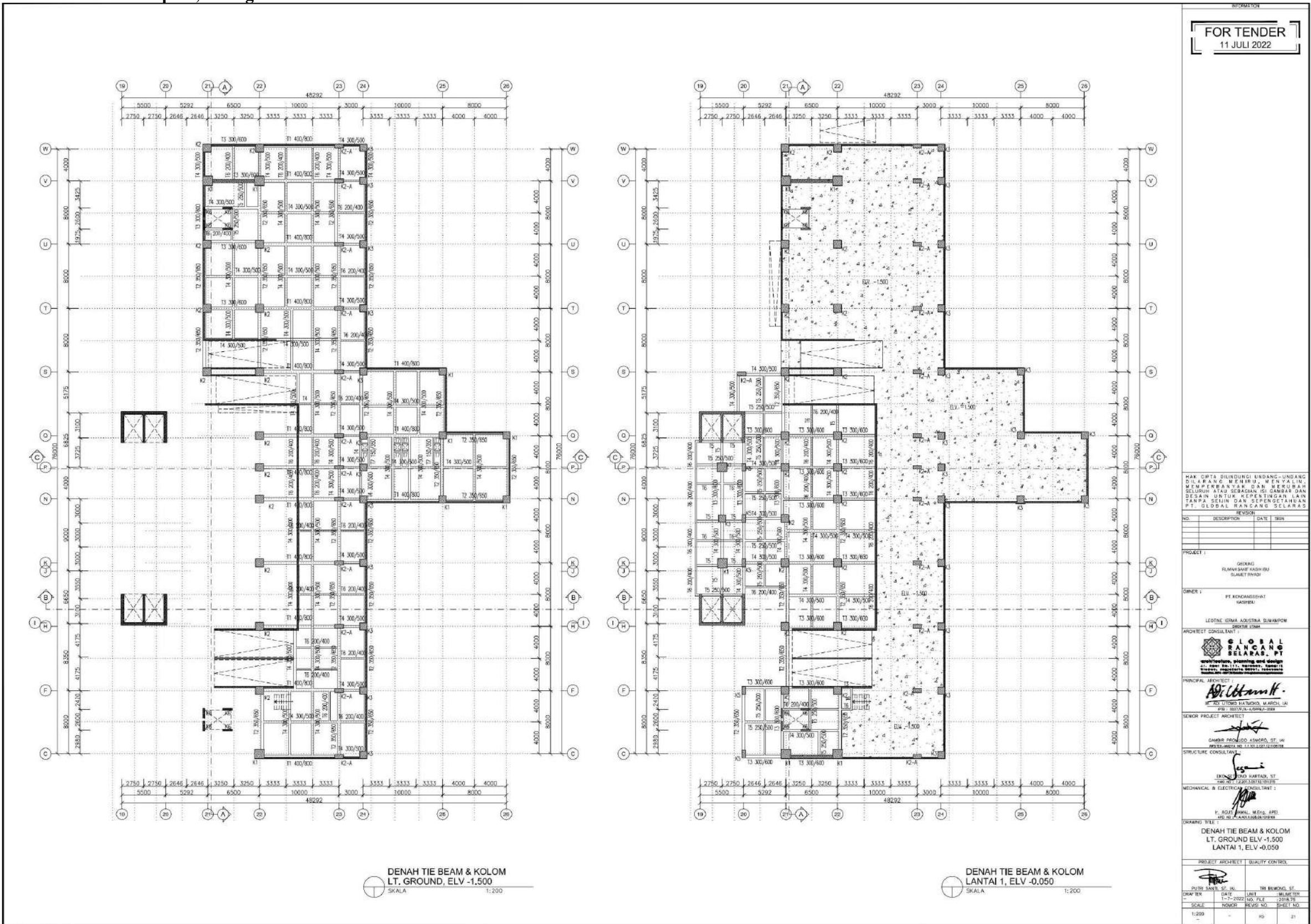
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KODE	K1		K2		K3		K4		K5		K6		K7		K8	
POSISI	TUMPUAN	LAPANGAN	TUMPUAN	LAPANGAN	TUMPUAN	LAPANGAN	TUMPUAN	LAPANGAN	TUMPUAN	LAPANGAN	TUMPUAN	LAPANGAN	TUMPUAN	LAPANGAN	TUMPUAN	LAPANGAN
POTONGAN																
DIMENSI	1200 X 1200	1200 X 1200	1000 X 1000	1000 X 1000	900 X 900	900 X 900	800 X 800	800 X 800	450 X 1000	450 X 1000	200 X 1200	200 X 1200	300 X 300	300 X 300	200 X 200	200 X 200
TULANGAN	56 D 22	56 D 22	36 D 22	36 D 22	28 D 22	28 D 22	20 D 22	20 D 22	40 D 22	40 D 22	16 D 16	16 D 16	8 D 13	8 D 13	8 P 12	8 P 12
SENGKANG	4D10-150	4D10-200	4D10-150	4D10-200	4D10-100	4D10-150	4D10-150	4D10-200	4D10-150	4D10-200	2D10-150	2D10-200	2P8-150	2P8-200	2P6-150	2P6-200

DIMENSI DAN PENULANGAN KOLOM										
KODE	K2-A		K3-A		K3-B		K4-A		K4-B	
POSISI	TUMPUAN	LAPANGAN	TUMPUAN	LAPANGAN	TUMPUAN	LAPANGAN	TUMPUAN	LAPANGAN	TUMPUAN	LAPANGAN
POTONGAN										
DIMENSI	500 X 1100	500 X 1100	500 X 900	500 X 900	500 X 900	500 X 900	500 X 800	500 X 800	500 X 800	500 X 800
TULANGAN	26 D 22	26 D 22	20 D 22	20 D 22	24 D 22	24 D 22	16 D 22	16 D 22	24 D 22	24 D 22
SENGKANG	3D10-150	3D10-200	3D10-100	3D10-150	3D10-100	3D10-150	3D10-150	3D10-200	3D10-150	3D10-200

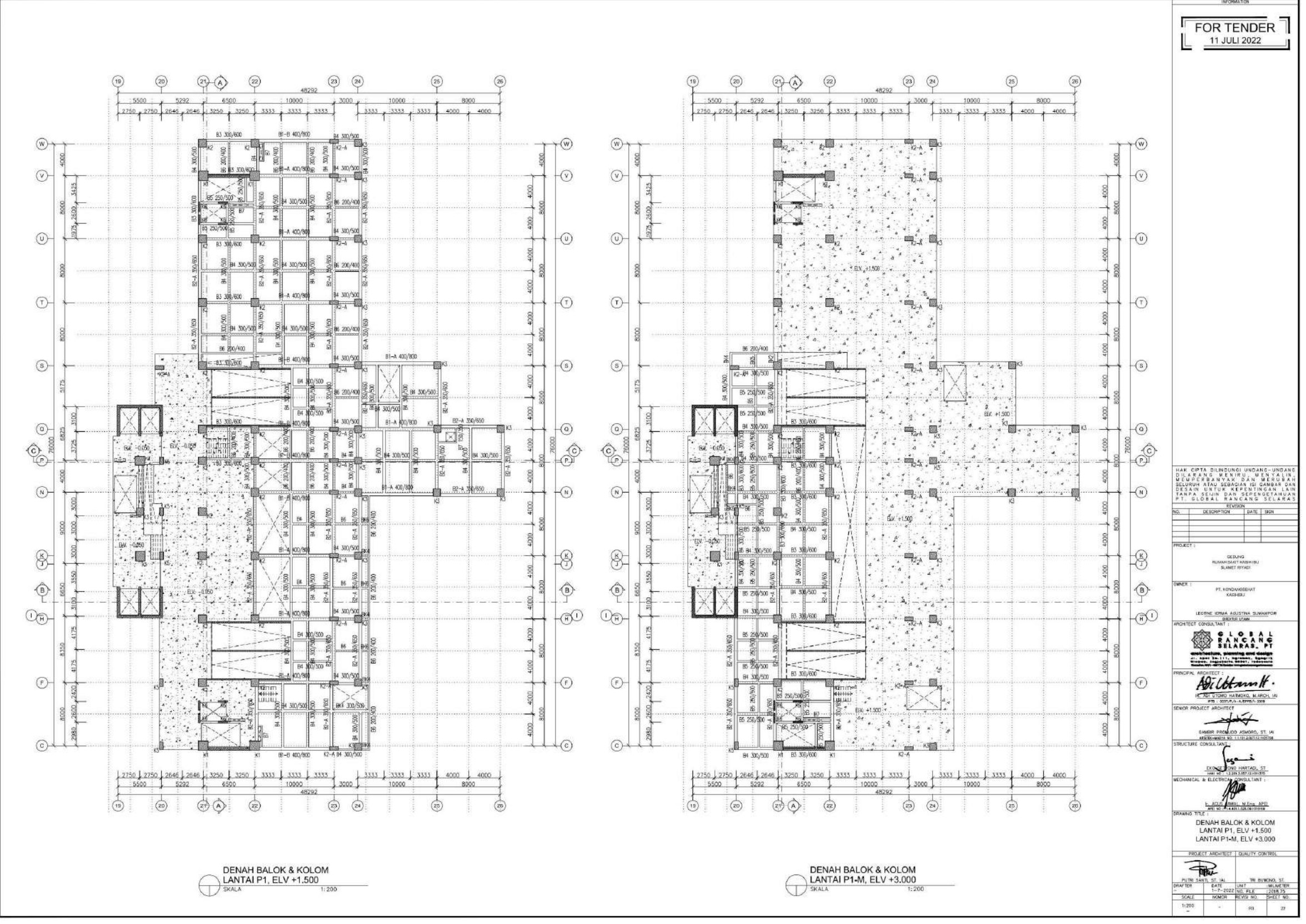
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INFORMATION			
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REVISI			
NO.	DESCRIPTION	DATE	SIGN
PROJECT : <div style="text-align: center; margin-top: 5px;"> GREKUNG RUMAH SAKIT KAGUR-KU SUMBER PINGER </div>			
OWNER : <div style="text-align: center; margin-top: 5px;"> PT. KONGKONGSEHET KANDUBU </div>			
LOCATION : JENAS AGUSTINA SUMAMPOR ARKHEOLOGIS : SUMBER STANA			
<div style="display: inline-block; vertical-align: middle; text-align: left;"> PT. GLOBAL RANCANG SELARAS. PT <i>architectural, planning and design</i> <small> AL : ARIE KUSUMITR, MScEng, SpArch Principal Architect PR : KONGKONGSEHET, SpArch </small> </div>			
PRINCIPAL ARCHITECT : <div style="text-align: center; margin-top: 5px;"> Arie Kusumitri PT. ARIE KUSUMITRI, MScEng, SpArch </div>			
SENIOR PROJECT ARCHITECT <div style="text-align: center; margin-top: 5px;"> DAMAR PRIYANTO ASWORO, ST. IN ARCHITECT - INDIA NO. 1.131.0.022.03.000000 </div>			
STRUCTURE CONSULTANT : <div style="text-align: center; margin-top: 5px;"> EKO NUGROHO HARTONO, ST. IND. NO. 1.3.001.000.03.000000 </div>			
MECHANICAL & ELECTRICAL CONSULTANT : <div style="text-align: center; margin-top: 5px;"> A. ADNAN JAMAL, M.Eng, AEC IND. NO. 1.4.001.000.03.000000 </div>			
DRAWING TITLE : <div style="text-align: center; margin-top: 10px;"> DETAIL PENULANGAN BALOK, KOLOM, THE BEAM & PLAT LANTAI </div>			
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DRAFTER	DATE	UNIT	MILIMETER
	11-7-2022	IND. FILE	0016.75
SCALE	NOVOR	REVISI NO.	SHEET NO.
NTS	-	RS	16

Appendix 2 Beam and Column plan, floor ground and 1.



Appendix 3 Beam and Column plan, floor P1 and P1-m.



FOR TENDER
11 JULI 2022

HAK CIPTA DILINDUNGI UNDANG-UNDANG
DILARANG MENIRU, MENYALIN,
MEMPERBANYAK, DAN MEMUBLIKASIKAN
SELURUH ATAU SEBAGIAN ISI GAMBAR DAN
DOKUMEN INI TANPA IZIN MENYERTA
TANPA SEJEN DAN SEPENGETAHUAN
PT. GLOBAL RANCANG SELARAS

NO.	REVISION	DESCRIPTION	DATE	SKRIPSI

PROJECT :
RUMAH SANT KASIH (RUKAS)
SUMATRA UTARA

OWNER :
PT. KONDAGSEWAT
KAB. SERDANG

LEONIE, CINDA, AGUSTINA, SILVAMPOR
SINERGI SORANG

ARCHITECT CONSULTANT :
GLOBAL RANCANG SELARAS, PT
Architecture, planning and design
Jl. Raya P. 11, No. 11, Medan, 56001
Telp. (061) 4211111, 4211112, 4211113
Email: global@selaras.com, info@selaras.com

PRINCIPAL ARCHITECT :
Asri Utami H.
NIP. 198003011990001, NIP. 198003011990002
NIP. 198003011990003

SENIOR PROJECT ARCHITECT :
[Signature]

GAMBAR PROJEKSI KAWASAN, ST. IN
KAWASAN KAWASAN KAWASAN KAWASAN

STRUCTURE CONSULTANT :
[Signature]

EXCISE, ZONO HARTAD, ST
KAWASAN KAWASAN KAWASAN KAWASAN

MECHANICAL & ELECTRICAL CONSULTANT :
[Signature]

DRIVING TITLE :
DENAH BALOK & KOLOM
LANTAI P1, ELV +1.500
LANTAI P1-M, ELV +3.000

PROJECT ARCHITECT	QUALITY CONTROL
[Signature]	[Signature]

DRAWN	DATE	UNIT	REVISION
PT. NI SANS, ST. IN	1-7-2022	NO. FILE	2018.25

SCALE	REVISION	NO.	SHEET NO.
1:200			

[illegible]

DENAH BALOK & KOLOM-M
LANTAI P2, ELV +4.500
SKALA 1:200

DENAH BALOK & KOLOM
LANTAI P2-M, ELV +6.000
SKALA 1:200

HAK CIPTA SUNDHI UNDANG-UNDANG
DILAKUKAN MENJUAL, MENYALIN,
MEMPERBANYAK, DAN MERUBAH
SELURUH ATAU SEBAGIAN ISI GAMBAR DAN
BIDANG UNTUK KEPERINGATAN LAIN
TANPA SELIN DAN SEPENGANTARAN
PT. GLOBAL RANCANG SELARAS

NO.	DESCRIPTION	DATE	SIGN
1	REVISI		

PROJECT :
Gedung
RUMAH SAKIT/RSU
SUKSES (RSU)

OWNER :
PT. KONGRESIAWATI
KARASIA

LECTURE :
ENNA AGUSTINA, SUMAMPO
SUKSES (RSU)

ARCHITECT CONSULTANT :
GLOBAL RANCANG SELARAS, PT
architect, planning and design
for all types of building, civil
engineering, landscape, interior
design, and other related services

PRINCIPAL ARCHITECT :
[Signature]
ARCHITECT (S) / SUTISNO, M. ARCH. (SI)
IPB - SURABAYA / 2013 / 2014

SENIOR PROJECT ARCHITECT :
[Signature]
ARCHITECT (S) / SUTISNO, M. ARCH. (SI)
IPB - SURABAYA / 2013 / 2014

STRUCTURE CONSULTANT :
[Signature]
DR. HENDRIK HARTONO, ST., MT
IPB - SURABAYA / 2013 / 2014

MECHANICAL & ELECTRICAL CONSULTANT :
[Signature]
Y. ALI, M.Eng. (M.Eng. APT)
IPB - SURABAYA / 2013 / 2014

DRAWING TITLE :
DENAH BALOK & KOLOM
LANTAI P2, ELV +4.500
LANTAI P2-M, ELV +6.000

PROJECT ARCHITECT	QUALITY CONTROL
<i>[Signature]</i>	<i>[Signature]</i>

DATE : 1-7-2022
SCALE : 1:200
SHEET NO : 01

PROJECT ARCHITECT :
DR. HENDRIK HARTONO, ST., MT
IPB - SURABAYA / 2013 / 2014

QUALITY CONTROL :
Y. ALI, M.Eng. (M.Eng. APT)
IPB - SURABAYA / 2013 / 2014

DRAWING TITLE :
DENAH BALOK & KOLOM
LANTAI P2, ELV +4.500
LANTAI P2-M, ELV +6.000

PROJECT ARCHITECT	QUALITY CONTROL
<i>[Signature]</i>	<i>[Signature]</i>

DATE : 1-7-2022
SCALE : 1:200
SHEET NO : 01

FOR TENDER
11 JULI 2022

DENAH BALOK & KOLOM
LANTAI P3, ELV +7.500

SKALA 1:200

DENAH BALOK & KOLOM
LANTAI P3-M, ELV +9.000

SKALA 1:200

HAK CIPTA BILINDUNG UNDAK-UNDANG
DIPATRI WISNU, WISNU
MEMPERBANYAK DAN MERUBAH
SELURUH ATAU SEBAGIAN DARI GAMBAR
DESAIN UNTUK KEPENTINGAN LAIN
TANPA SELIN DAN PERSETUJUAN
PT. GLOBAL RANGKAI SELARAS

REVISI

NO.	DESCRIPTION	DATE	SIN

PROJECT :

REKONSTRUKSI DAN PERUBAHAN
RUMAH SAKIT RAKAH BUI
SUMBER RAKAH

OWNER :

PT. KONGKONGKONG
KARAHU

LEADLINE: ERMA AGUSTINA SUMAPITUR
INCHONG KONG

ARCHITECT CONSULTANT :

GLOBAL RANGKAI SELARAS, PT

ARCHITECT, PLANNING AND DESIGN
CONSULTANT
REKONSTRUKSI DAN PERUBAHAN
RUMAH SAKIT RAKAH BUI
SUMBER RAKAH

PRINCIPAL ARCHITECT

PT. KONGKONGKONG, KARAHU

SENIOR PROJECT ARCHITECT

GAMBAR PROJEKSI ARSITEKTUR, ST. HAI
ARHITEK, MERSI, ST. HAI

STRUCTURE CONSULTANT :

PT. KONGKONGKONG, KARAHU

MECHANICAL & ELECTRICAL CONSULTANT :

PT. KONGKONGKONG, KARAHU

DRAWING TITLE :

DENAH BALOK & KOLOM

LANTAI P3, ELV +7.500

LANTAI P3-M, ELV +9.000

PROJECT ARCHITECT QUALITY CONTROL

PT. KONGKONGKONG, KARAHU

PT. KONGKONGKONG, KARAHU

PT. KONGKONGKONG, KARAHU

PT. KONGKONGKONG, KARAHU

PT. KONGKONGKONG, KARAHU

PT. KONGKONGKONG, KARAHU

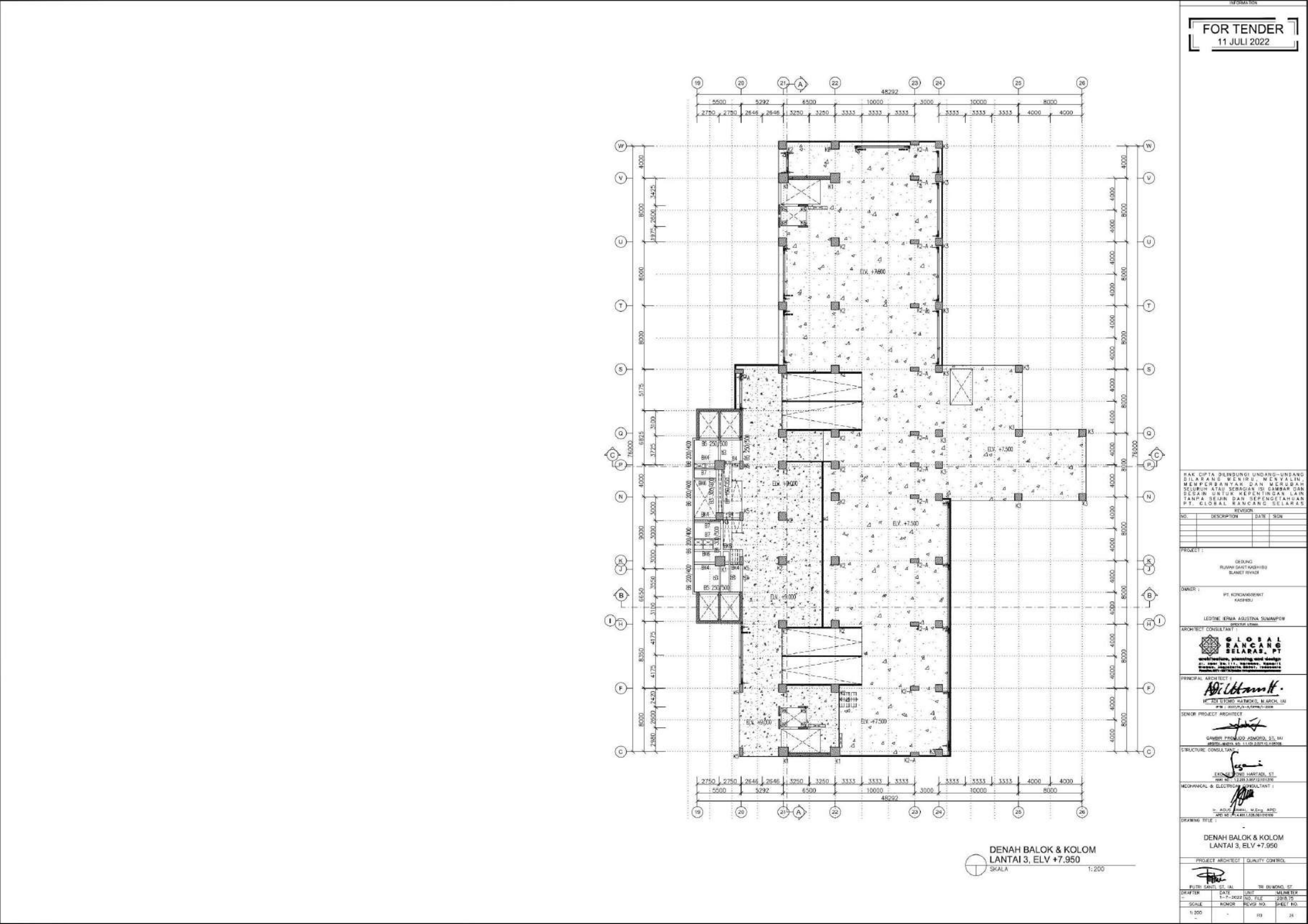
PT. KONGKONGKONG, KARAHU

PT. KONGKONGKONG, KARAHU

PT. KONGKONGKONG, KARAHU

PT. KONGKONGKONG, KARAHU

Appendix 7 Beam and Column plan, floor 3th



FOR TENDER
11 JULI 2022

DENAH BALOK & KOLOM
LANTAI P4, ELV +10.500
SKALA 1:200

DENAH BALOK & KOLOM
LANTAI P4-M, ELV +12.000
SKALA 1:200

HAK CIPTA SENGUNDI UNDANG-UNDANG
DILARANG MELAKUKAN MENYALIN,
MEMPERSENGKAN, MERUBAH
SELURUH ATAU SEBAGIAN ISI GAMBAR DAN
DESAIN UNTUK KEPERSTINGGAN LAIN
TANPA SELIN DAN PERSETUJUAN
PT. GLOBAL RANCANG SELARAS.

NO.	REVISI	DESCRIPTION	DATE	SIGN

PROJECT :
Gedung
RUMAH SAKIT RACHIBU
SLAMET RWADI

OWNER :
PT. KONGRESDEWAT
(KADHE)

LEAD ARCHITECT:
LEOTINE ERMA AUSTINA SUNAMPON

ARCHITECT CONSULTANT :
GLOBAL RANCANG SELARAS, PT
architect, planning and design
for civil, building, landscape
interior, industrial design, renovation
and construction management

PRINCIPAL ARCHITECT:
[Signature]
NOMOR STRUKTUR MARCH 19
PIS : 010/P/MS/0002-0003

SENIOR PROJECT ARCHITECT:
[Signature]
CAMBER PRISONED ASHORO, ST. IN
PROFESSOR, NIDUS, NIDUS, NIDUS

STRUCTURE CONSULTANT:
[Signature]
DR. HENDRO HARTONO, ST.
PH.D. IN STRUCTURAL ENGINEERING

MECHANICAL & ELECTRONIC CONSULTANT:
[Signature]
H. AGUSMAN, M.Sc., APRI
PELIK, KAWULAN, KAWULAN

DRAWING TITLE:
DENAH BALOK & KOLOM
LANTAI P4, ELV +10.500
LANTAI P4-M, ELV +12.000

PROJECT ARCHITECT QUALITY CONTROL:
[Signature]

DESIGNED BY:	TRI BUNWUD, ST.
CHECKED BY:	UNIT
DATE:	1-7-2022
SCALE:	NOD. FILE - 2018 JK
1:200	SCHEMATIC NO. SHEET NO.

FOR TENDER
11 JULI 2022

DENAH BALOK & KOLOM
LANTAI P5, ELV +13.500
SKALA 1:200

DENAH BALOK & KOLOM
LANTAI P5-M, ELV +15.000
SKALA 1:200

HAK CIPTA SUNDUNG UNDANG-UNDANG
DI LAKUKAN MELIPUTI: MERUBAH,
MEMPERBAIKI, DAN MERUBAH
SELENGKAP ATAU BERGASAL SILOKAR DAN
DESAIN UNTUK KEPENTINGAN LAIN
TANPA SEJIN DAN SEPENGETAHUAN
PT. GLOBAL RANGKANG SELARAS

NO.	REVISI	DESCRIPTION	DATE	SKN
1				
2				
3				
4				
5				

PROJECT :

OWNER :

ARCHITECT CONSULTANT :

PRINCIPAL ARCHITECT :

SENIOR PROJECT ARCHITECT :

STRUCTURE CONSULTANT :

MECHANICAL & ELECTRICAL CONSULTANT :

DRAWING TITLE :

PROJECT ARCHITECT : QUALITY CONTROL :

DATE :

SCALE :

NO. SHEET :

PROJECT :

DATE :

SCALE :

DENAH BALOK & KOLOM
LANTAI 5, ELV +15.950
 SKALA 1:200

INFORMATION			
FOR TENDER			
11 JULI 2022			
HAK CPTA BILUNDUNG UNDANG-UNDANG			
SILABANG MERIPU, MESTYAL			
MEMPERBANYAK DAN MERUBAH			
SELURUH ATAU SEBAGIAN DARI GAMBAR DAN			
DESAIN UNTUK KEPERTINGIN LAIN			
TANPA SELIN DAN SEPENSTARAAN			
PT. GLOBAL RANCANG SELARAS			
REVISION			
NO	DESCRIPTION	DATE	SIGN
PROJECT :			
GEORING			
KAWAN SANTANAN RUI			
SUMBER BAYAN			
OWNER :			
PT. KONGKONGKAT			
(KAWAN)			
LOCATION: JALAN AGUSTIN, SUMANPOW			
SUMBER BAYAN			
ARCHITECT CONSULTANT :			
GLOBAL RANCANG SELARAS, PT.			
architectural, planning and design			
11, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22, 23, 24, 25, 26, 27, 28, 29, 30, 31, 32, 33, 34, 35, 36, 37, 38, 39, 40, 41, 42, 43, 44, 45, 46, 47, 48, 49, 50, 51, 52, 53, 54, 55, 56, 57, 58, 59, 60, 61, 62, 63, 64, 65, 66, 67, 68, 69, 70, 71, 72, 73, 74, 75, 76, 77, 78, 79, 80, 81, 82, 83, 84, 85, 86, 87, 88, 89, 90, 91, 92, 93, 94, 95, 96, 97, 98, 99, 100, 101, 102, 103, 104, 105, 106, 107, 108, 109, 110, 111, 112, 113, 114, 115, 116, 117, 118, 119, 120, 121, 122, 123, 124, 125, 126, 127, 128, 129, 130, 131, 132, 133, 134, 135, 136, 137, 138, 139, 140, 141, 142, 143, 144, 145, 146, 147, 148, 149, 150, 151, 152, 153, 154, 155, 156, 157, 158, 159, 160, 161, 162, 163, 164, 165, 166, 167, 168, 169, 170, 171, 172, 173, 174, 175, 176, 177, 178, 179, 180, 181, 182, 183, 184, 185, 186, 187, 188, 189, 190, 191, 192, 193, 194, 195, 196, 197, 198, 199, 200, 201, 202, 203, 204, 205, 206, 207, 208, 209, 210, 211, 212, 213, 214, 215, 216, 217, 218, 219, 220, 221, 222, 223, 224, 225, 226, 227, 228, 229, 230, 231, 232, 233, 234, 235, 236, 237, 238, 239, 240, 241, 242, 243, 244, 245, 246, 247, 248, 249, 250, 251, 252, 253, 254, 255, 256, 257, 258, 259, 260, 261, 262, 263, 264, 265, 266, 267, 268, 269, 270, 271, 272, 273, 274, 275, 276, 277, 278, 279, 280, 281, 282, 283, 284, 285, 286, 287, 288, 289, 290, 291, 292, 293, 294, 295, 296, 297, 298, 299, 300, 301, 302, 303, 304, 305, 306, 307, 308, 309, 310, 311, 312, 313, 314, 315, 316, 317, 318, 319, 320, 321, 322, 323, 324, 325, 326, 327, 328, 329, 330, 331, 332, 333, 334, 335, 336, 337, 338, 339, 340, 341, 342, 343, 344, 345, 346, 347, 348, 349, 350, 351, 352, 353, 354, 355, 356, 357, 358, 359, 360, 361, 362, 363, 364, 365, 366, 367, 368, 369, 370, 371, 372, 373, 374, 375, 376, 377, 378, 379, 380, 381, 382, 383, 384, 385, 386, 387, 388, 389, 390, 391, 392, 393, 394, 395, 396, 397, 398, 399, 400, 401, 402, 403, 404, 405, 406, 407, 408, 409, 410, 411, 412, 413, 414, 415, 416, 417, 418, 419, 420, 421, 422, 423, 424, 425, 426, 427, 428, 429, 430, 431, 432, 433, 434, 435, 436, 437, 438, 439, 440, 441, 442, 443, 444, 445, 446, 447, 448, 449, 450, 451, 452, 453, 454, 455, 456, 457, 458, 459, 460, 461, 462, 463, 464, 465, 466, 467, 468, 469, 470, 471, 472, 473, 474, 475, 476, 477, 478, 479, 480, 481, 482, 483, 484, 485, 486, 487, 488, 489, 490, 491, 492, 493, 494, 495, 496, 497, 498, 499, 500, 501, 502, 503, 504, 505, 506, 507, 508, 509, 510, 511, 512, 513, 514, 515, 516, 517, 518, 519, 520, 521, 522, 523, 524, 525, 526, 527, 528, 529, 530, 531, 532, 533,			

DENAH BALOK & KOLOM
LANTAI P6, ELV +16.500
SKALA: 1:200

DENAH BALOK & KOLOM
LANTAI P6-M, ELV +17.900
SKALA: 1:200

FOR TENDER
11 JULI 2022

NO.	DESCRIPTION	DATE	BY
1	REVISION		
2			
3			

PROJECT : GEDUNG RUMAH SAKIT KAH RI SURABAYA

OWNER : PT. KORDA WISATA HARIBU

LEAD ARCHITECT : LUTHER ERNA AGUSTINA SUMAPRAWATI

ARCHITECT CONSULTANT : **GLOBAL RANCANG SELARAS, PT.**
architectural, planning and design

PRINCIPAL ARCHITECT : **Adi Utami**
PT. KORDA WISATA HARIBU, SURABAYA

SENIOR PROJECT ARCHITECT : [Signature]

STRUCTURE CONSULTANT : [Signature]

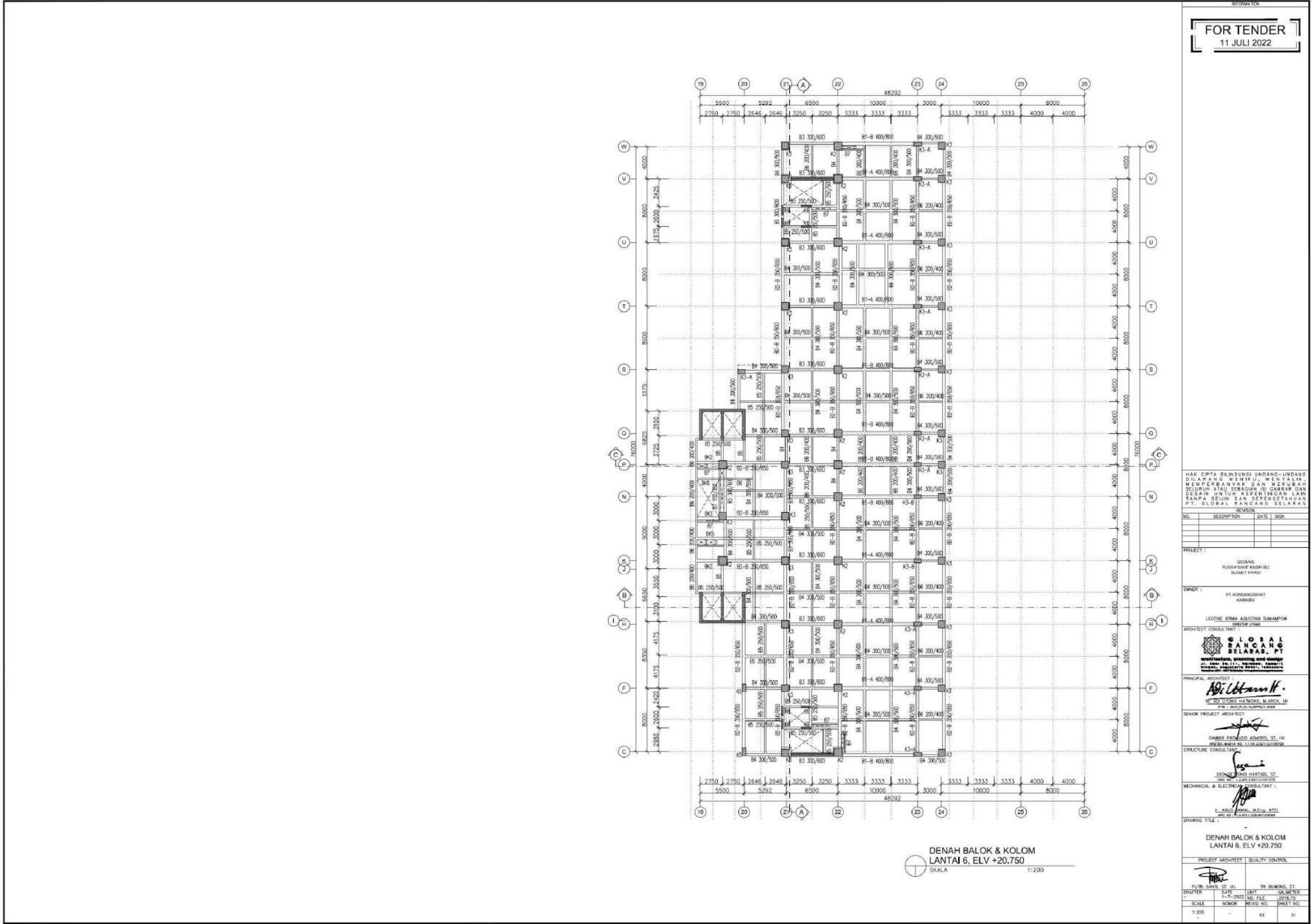
MECHANICAL & ELECTRICAL CONSULTANT : [Signature]

DRAWING TITLE : **DENAH BALOK & KOLOM LANTAI P6, ELV +16.500**

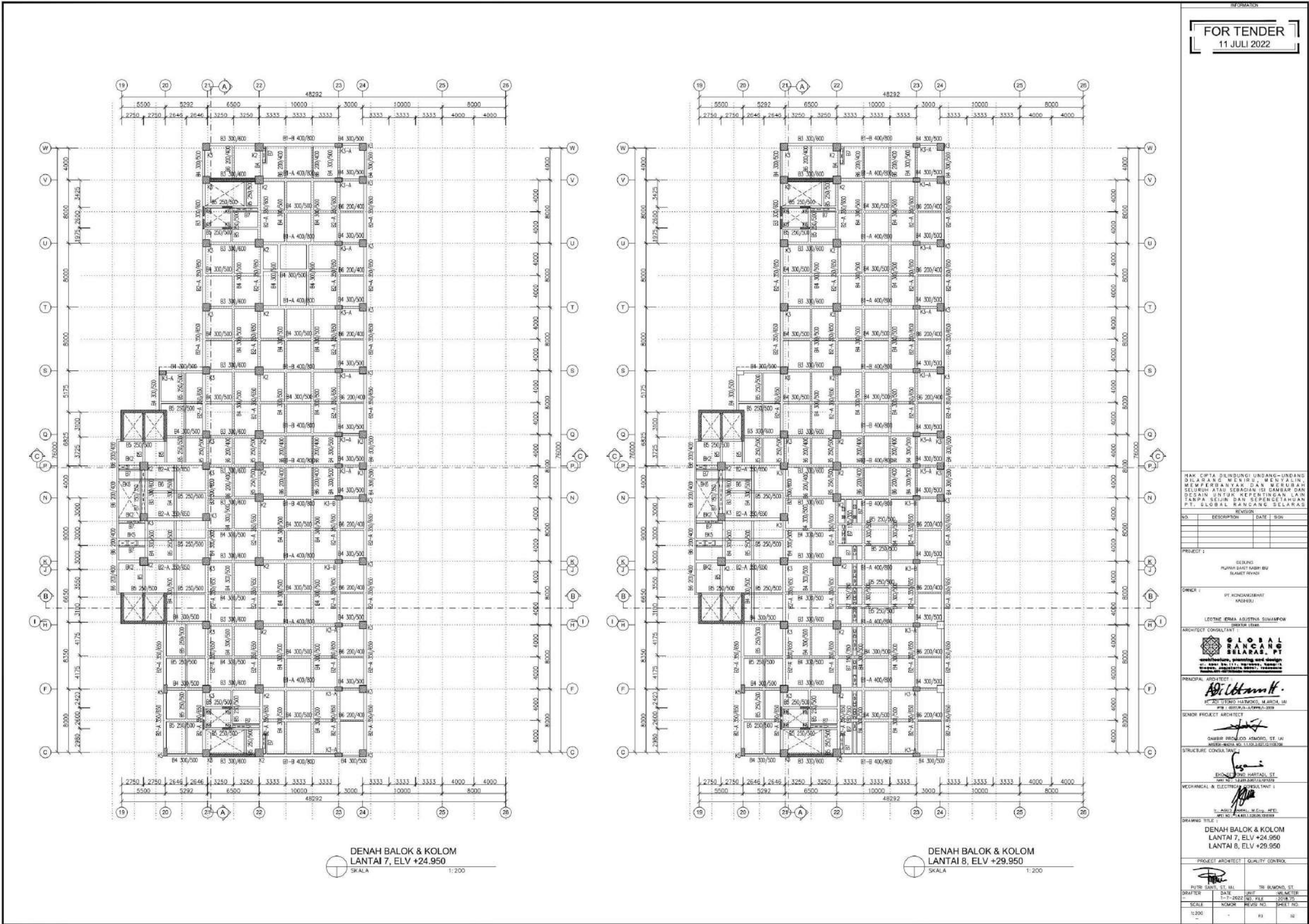
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DRFTER	DATE	UNIT	NUMBER
PUTRI BAYU, S.T.	10-7-2022	MM	2418/75
SCALE	1:200	NO. FILE	2418/75

Appendix 12 Beam and Column plan, 6th floor.



Appendix 13 Beam and Column plan, 7th floor and 8th floor.



FOR TENDER
11 JULI 2022

DENAH BALOK & KOLOM
LANTAI 9, ELV +34.150
SKALA 1:200

DENAH BALOK & KOLOM
LANTAI 10, ELV +38.350
SKALA 1:200

HAZ CETA DILINEUR UNDANG-UNDANG
DIARANG MENIRU, MENYALIN,
MEMPERBANYAK, DAN MERUBAH
SELURUH ATAU SEBAGIAN DI GAMBAR DAN
DESAIN UNTUK KEPENTINGAN LAIN
TANPA SEJUNTAAN SEMENTARA
PT GLOBAL RANGKANG SELARAS

NO.	DESCRIPTION	DATE	SIGN
1	REVISION		
2			
3			

PROJECT :
Gedung
RUMAH SAKIT GASH BSI
BLANDETREK

OWNER :
PT. KONGSANGHAT
KABERUS

ARCHITECT CONSULTANT :
LEOTINE PERMA AGUSTINA SUMAPON
RANGKANG SELARAS, PT

PRINCIPAL ARCHITECT :
Agustina Perma
PT. PERMA AGUSTINA SUMAPON, IN
PT. RANGKANG SELARAS, PT

SENIOR PROJECT ARCHITECT :
GABRIEL PRIMOLO JUMBO, ST. (A)
RANGKANG SELARAS, PT

STRUCTURE CONSULTANT :
EKO PRIMO PARTO, ST. (A)
RANGKANG SELARAS, PT

MECHANICAL & ELECTRICAL CONSULTANT :
P. ALVIN RAMA, M. Eng. APD
RANGKANG SELARAS, PT

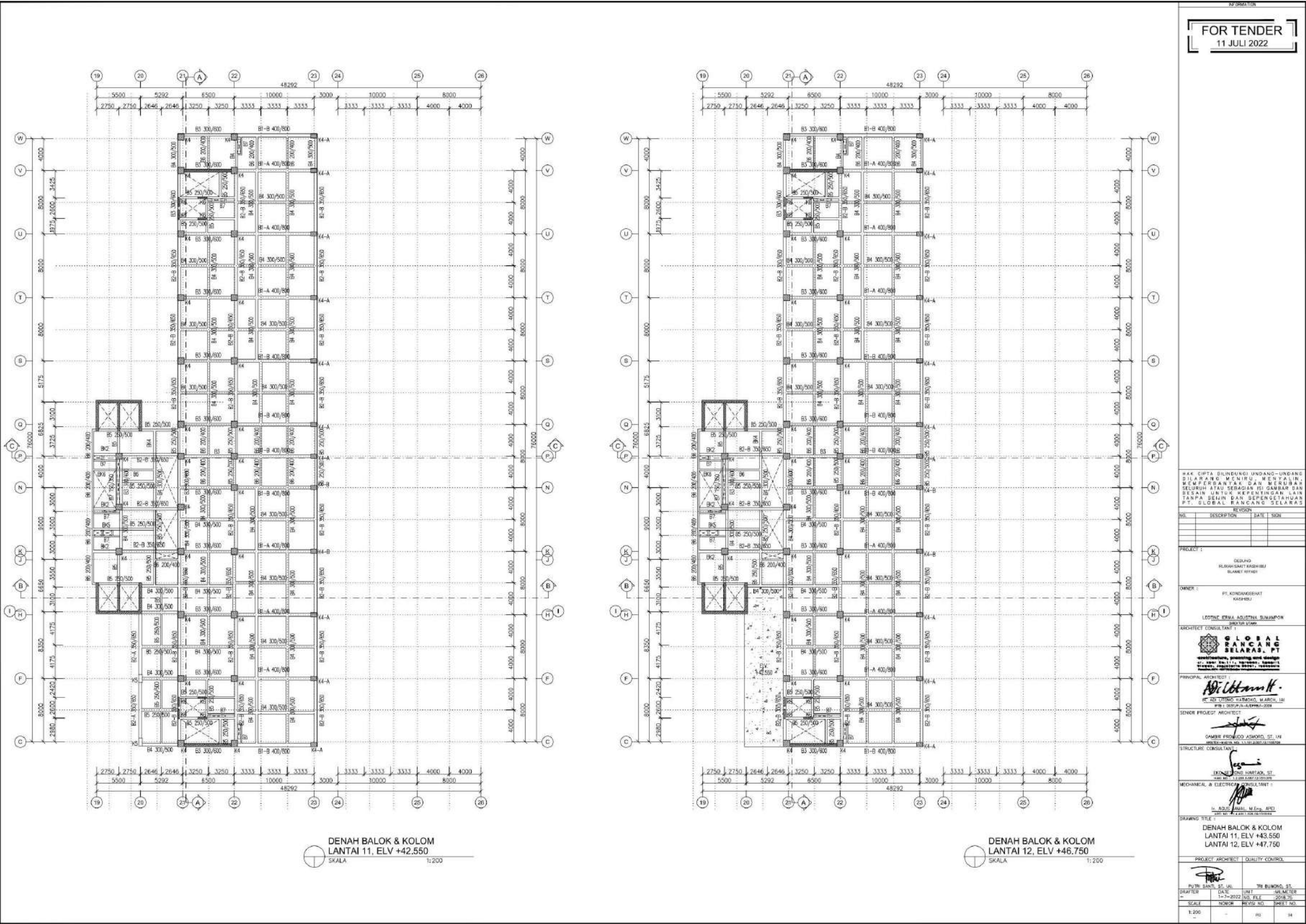
DRAWING TITLE :
DENAH BALOK & KOLOM
LANTAI 9, ELV +34.150
LANTAI 10, ELV +38.350

PROJECT ARCHITECT :
Agustina Perma
QUALITY CONTROL :
GABRIEL PRIMOLO JUMBO

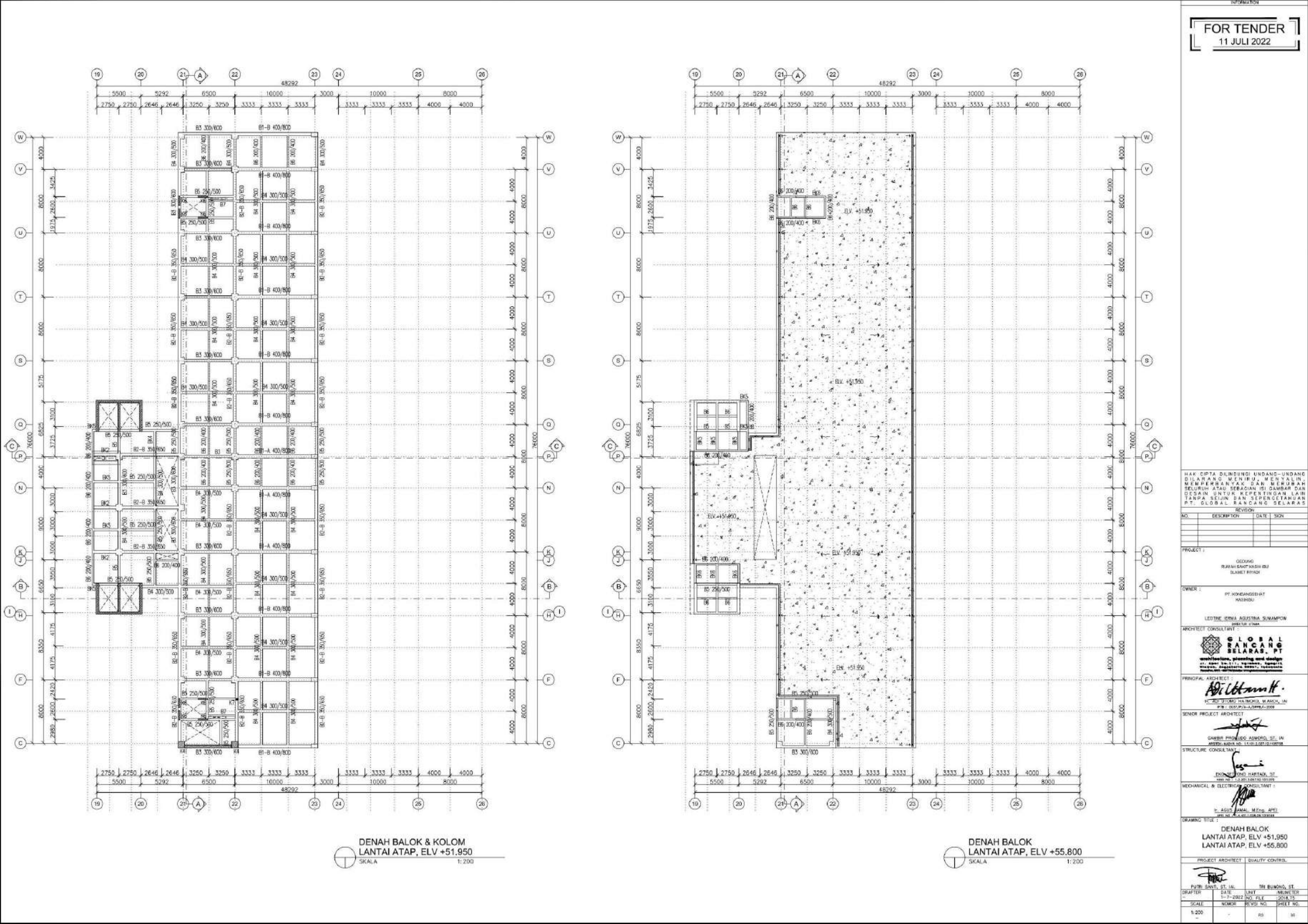
DATE :
11-7-2022
SCALE :
1:200

REVISION :
NO. 01
REVISI :
1

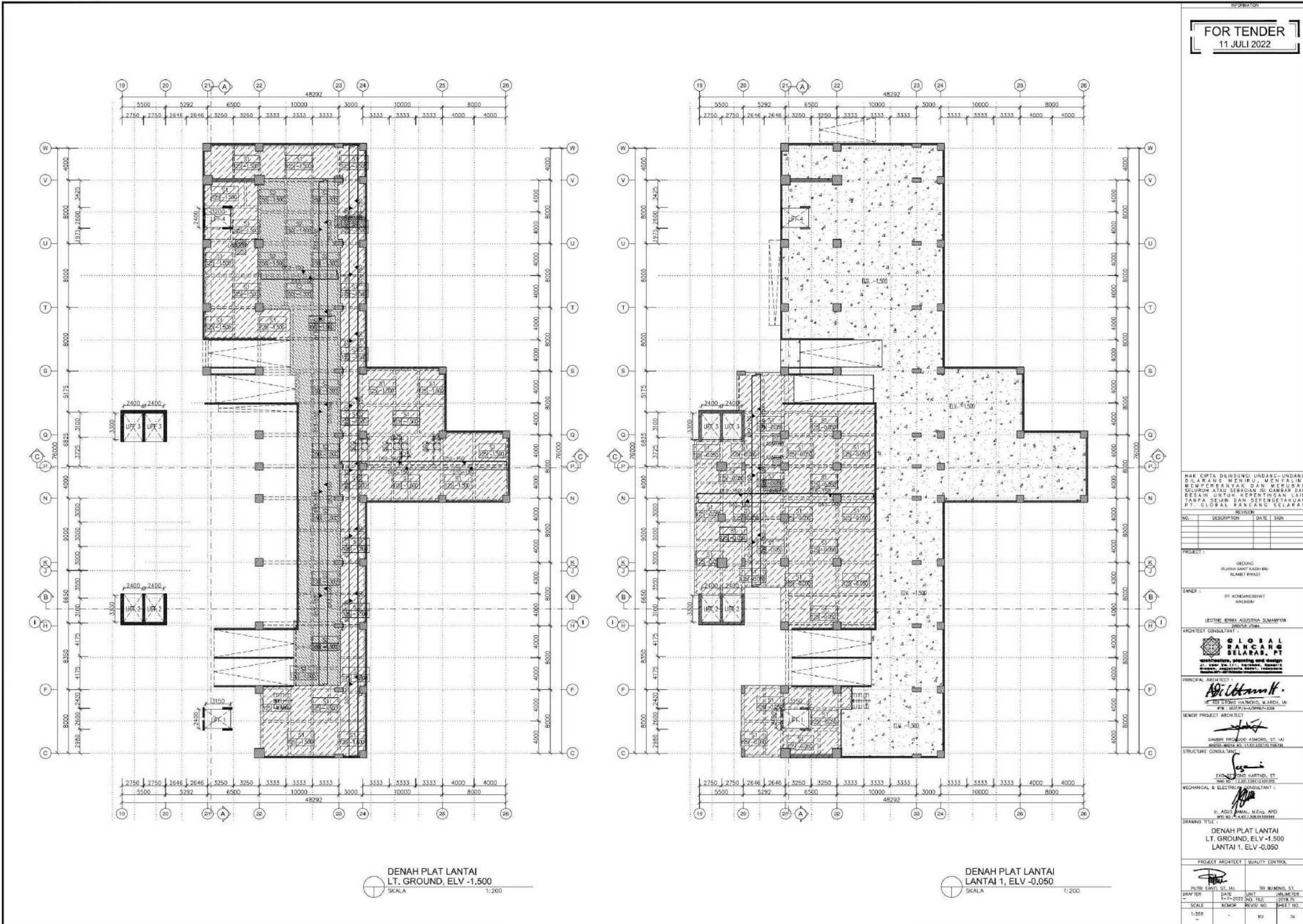
Appendix 15 Beam and Column plan, 11th floor and 12th floor.



Appendix 16 Beam and Column plan, Roof top.



Appendix 17 Floor slabs plan, ground and 1st floor.



FOR TENDER
11 JULI 2022

DENAH PLAT LANTAI
LANTAI P1, ELV +1.500
SKALA 1:200

DENAH PLAT LANTAI
LANTAI P1-M, ELV +3.000
SKALA 1:200

HAK CIPTA BILANGKUP UNDANG-UNDANG
DILARANG MENIRU, MENYALIN,
MEMPERKANYAKAN, MENYALIN,
SELOKAH ATAU SEBAGIAN ISI GAMBAR DAN
DESKRIPSI UNTUK KEPERLUAN LAIN
TANPA IZIN DAN SEPENGETAHUAN
PT. GLOBAL RANCANG SELARAS

NO.	REVISI	DESCRIPTION	DATE	SKRIP
1				
2				
3				
4				
5				

PROJECT :

OWNER :

ARCHITECT CONSULTANT :

PRINCIPAL ARCHITECT :

SENIOR PROJECT ARCHITECT :

STRUCTURE CONSULTANT :

MECHANICAL & ELECTRICAL CONSULTANT :

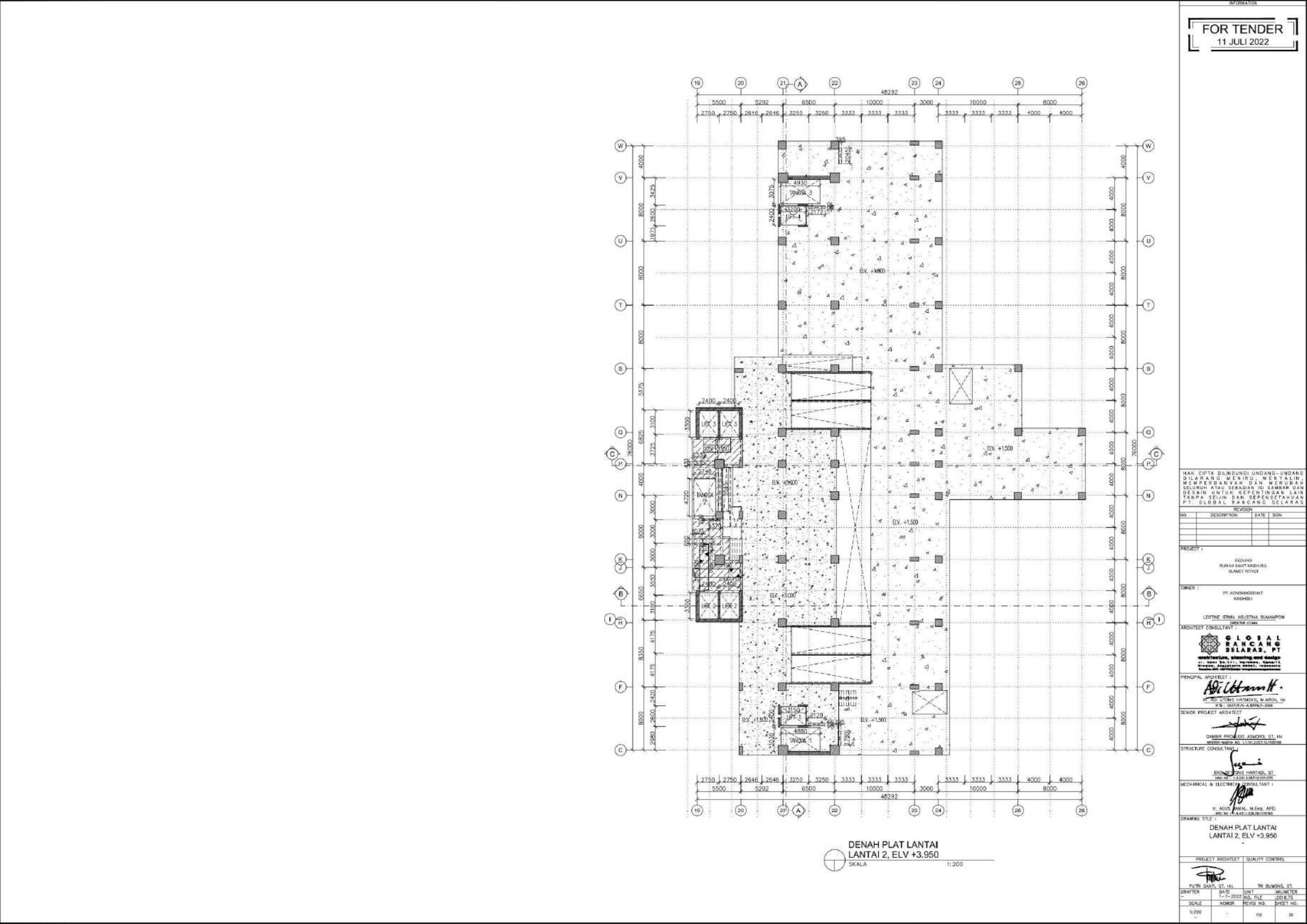
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PROJECT ARCHITECT :

QUALITY CONTROL :

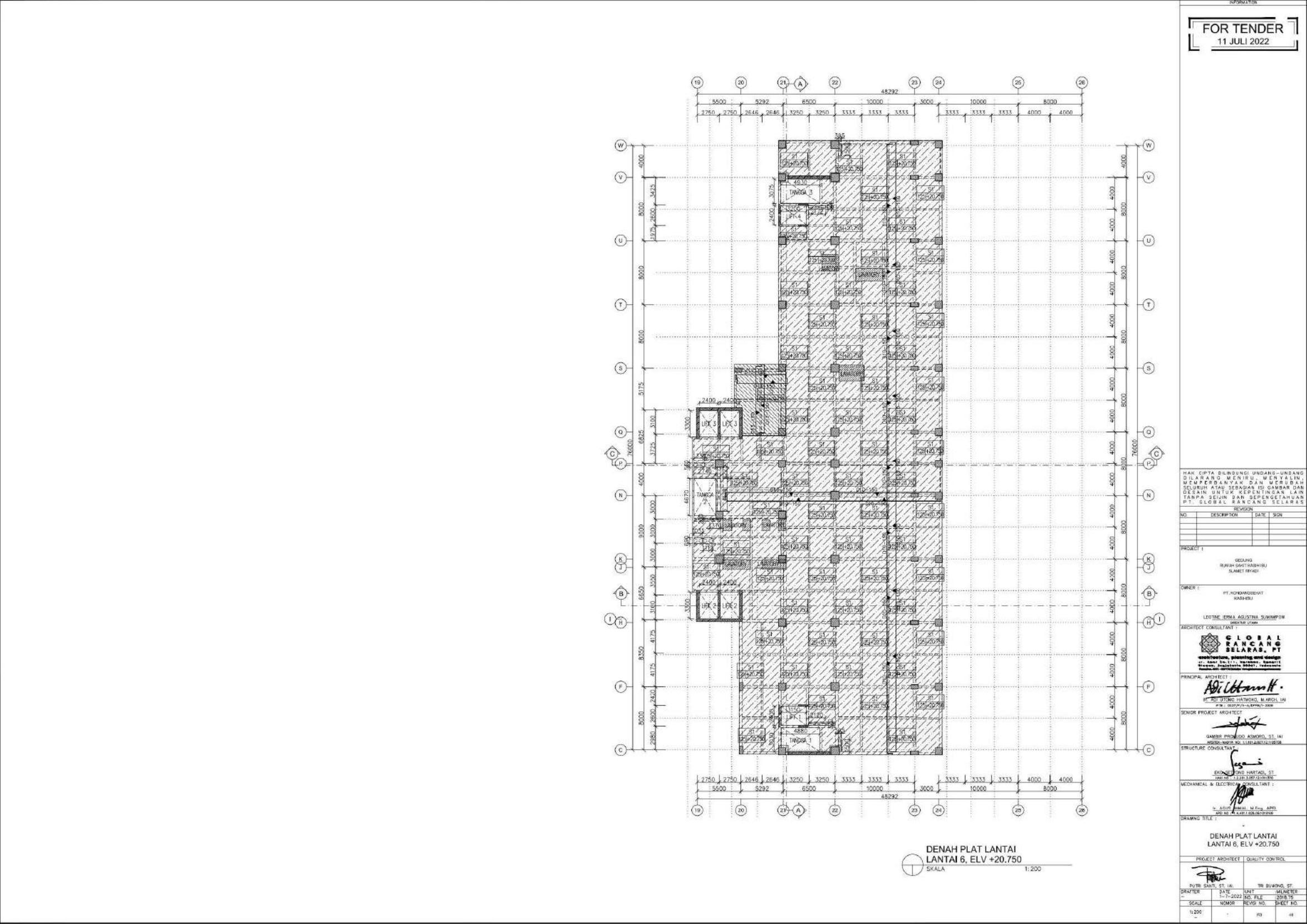
DATE :

Appendix 19 Floor slabs plan, 2nd floor.



[illegible]

Appendix 21 Floor slabs plan, 6th floor.



Appendix 22 Floor slabs plan, 7th and 8th floor.

