FINAL PROJECT

COMPARATIVE ANALYSIS BETWEEN INTERNAL FORCES OF FIXED AND FLEXIBLE FOUNDATION UNDER EARTHQUAKE LOAD

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

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2024

PLAGIARISM STATEMENT

I declare that the Final Assignment report I prepared as a requirement for completing the Bachelor's degree program in the Civil Engineering Undergraduate Program, Faculty of Civil Engineering and Planning, Universitas Islam Indonesia is the result of my work. Certain parts of writing the Final Project report that I quoted from other people's work have been written in the source clearly in accordance with the norms, rules, and ethics of writing scientific papers. If in the future it is discovered that all or part of this Final Project report is not my own work or there is plagiarism in certain parts, I am willing to accept sanctions, including revocation of the academic title I hold in accordance with applicable laws and regulations.

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

Tested on 09/01/2024
By the Board of Examiners

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ABSTRACT

Pile foundations can be designed as fixed or flexible according to the soil parameters, the seismic condition of the area, and the characteristics of the building. This research compares the effect of internal forces working on fixed and flexible foundations with the structure of a symmetrical 15-story building of reinforced concrete without a structural shear wall located in Yogyakarta, Indonesia with medium soil according to SNI 1726:2019.

This study analyzes the effect of the flexibility of pile foundations on the natural period of vibration of a building, compares the internal forces acting upon fixed and flexible pile foundations in medium soil, and reveals the effects of soil shear modulus distribution. The outcomes can be obtained after an extensive design that is generally divided into two stages of planning according to FEMA P-750 and P-751. After the structural planning, the analysis of the spring stiffness parameters of the flexible foundation is carried out, which is used to compare the internal forces results with the fixed foundation.

The result of the comparison analysis shows that the building with a flexible foundation has a longer fundamental period. The internal forces including base shear and column bending moment of the flexible foundation are relatively smaller than the fixed foundation, while the beam bending moment, column shear force, drift ratio, joint rotation, and horizontal joint displacement all show a higher value. Meanwhile, the effects of soil shear modulus distribution with a homogeneous soil profile show a higher value compared to the parabolic soil profile.

Keywords: Fixed Foundation, Flexible Foundation, Fundamental Period, Internal Forces, Soil Shear Modulus

FOREWORD

The author would like to thank Allah SWT for His guidance to complete the Final Project titled “Comparative Analysis Between Internal Forces of Fixed and Flexible Foundation Under Earthquake Load”. This Final Project is one of the academic requirements for completing the Civil Engineering Undergraduate Program, Faculty of Civil Engineering and Planning, Universitas Islam Indonesia, Yogyakarta.

In preparing this Final Project, the author faced several obstacles, but thanks to suggestions, criticism, and encouragement from various people, this Final Project was completed. In this regard, the author would like to express their deepest thanks to:

1. Prof. Ir. Widodo, MSCE., Ph.D. as the Supervisor,
2. Astriana Hardawati, S.T., M.Eng. as the Examiner I,
3. Ir. Yunalia Muntafi, S.T., M.T., Ph.D. (Eng)., IPM. as the Examiner II,
4. The author’s parents who have sacrificed so much both materially and spiritually for the author to complete this Final Project.

Finally, the author hopes that this Final Project will be useful for various people who read it.

Yogyakarta, January 2024
Author,

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LIST OF NOTATIONS AND ABBREVIATIONS

ELA = Equivalent Linear Approach

SSI = Soil-Structure Interaction

K_r = Pile relative stiffness

G = Shear modulus of soil

G_{max} = Maximum shear modulus of soil

SPT = Standard Penetration Test

CPT = Cone Penetration Test

L = Length

B = Width

H = Height

D = Diameter

α_f = Ratio of the bending stiffness of the beam section to the bending stiffness of the plate width limited laterally by the center line of the adjacent panel (if any) on each side of the beam

Ecb = Elastic modulus of the concrete beam (MPa)

Ecs = Elastic modulus of the concrete slab (MPa)

Ib = Moment of inertia of the beam gross cross-section about the central axis (mm^4)

Is = Moment of inertia of the slab gross cross-section about the central axis (mm^4)

a = Stairs *antrede* width

s = Stairs *optrede* height

α = Stairs tilt angle (°)

t1 = Equivalent thickness of stairs (cm)

Ie = Importance factor

Ss = The risk-targeted maximum considered earthquake ground motion (MCE_R) for the response spectrum of short periods (5% critical damping)

S1 = The risk-targeted maximum considered earthquake ground motion (MCE_R) for the response spectrum of 1 second period (5% critical damping)

Fa = Acceleration-related amplification factor representing short-period vibrations

Fv = Acceleration-related amplification factor representing vibrations of 1 second period

S_{MS} = Acceleration spectral response parameter at short periods

S_{M1} = Acceleration spectral response parameter at a period of 1 second

S_{DS} = Design spectral acceleration parameter for short periods

S_{D1} = Design spectral acceleration parameter for a period of 1 second

Sa = Acceleration response spectra

T = Period of the fundamental vibration of the structure (s)

T_L = Long-period transition of the fundamental vibration of the structure (s)

R = Response modification coefficient

Ω = System additional strength factor

Cd = Deflection magnification factor

Cu = Coefficient for calculated period upper bound

Ta = Approach fundamental period (s)

Cs = Seismic response coefficient

V = Seismic base shear force (kN)

V_t = Combined response for the base shear force resulting from the analysis of variance (kN)

W = Effective seismic weight

F_x = Lateral seismic force (kN)

C_{vx} = Vertical distribution factor

w_i or w_x = The portion of the total effective seismic weight of the structure (W) placed or imposed at story i or x

h_i or h_x = Height from base to story i or x (m)

k = The exponential value associated with the period of the structure

V_x = The design seismic story shear at all stories or at and above story x (kN)
F_i = The portion of the seismic shear force at story i (kN)
δ_i = Enlarged displacement at story i
δ_{ei} = Elastic displacement calculated due to the design seismic force at story i
Δ_i = Story drift at story i
Δ_a = Allowable story drift
h_{sx} = Story height below story x
θ = Stability coefficient
P_x = Total vertical design load at and above the story x (kN)
β = Ratio of the story shear requirement to shear capacity between story x and the story below
ρ = Redundancy factor
M_t = Inherent or natural torsional moment
M_{ta} = Accidental torsional moment
e = Total eccentricity
e_o = Natural eccentricity
e_t = Additional 5% torsional eccentricity
ELF = Equivalent Lateral Force
RSA = Response Spectrum Analysis
THA = Time History Analysis
SRSS = Square Root Sum of Squares
CQC = Complete Squares Combination
f’c = Compressive strength of concrete quality (MPa)
fy = Yield stress strength of profile steel quality (MPa)
$\gamma_{concrete}$ = Volume weight or specific gravity of concrete
P = Vertical load (kN)
Mx = Flexural moment at x direction
My = Flexural moment at y direction
Qp = End bearing capacity of pile foundation (ton)

Qs = Cover bearing capacity of pile foundation (ton)
Qu = Ultimate bearing capacity of pile foundation (ton)
Ap = End of pile area (m^2)
As = Cover of pile area (m^2)
SF = Safety Factor
Q_{all} = Allowable bearing capacity of pile foundation (ton)
Eg = Pile group efficiency
n = Number of pile foundations in x direction
m = Number of pile foundations in y direction
x_{max} = Maximum arm length of the pile in the x direction to the center of gravity of the pile cap
y_{max} = Maximum arm length of the pile in the y direction to the center of gravity of the pile cap
P_{max} = Pile foundation maximum axial force (ton)
P_{min} = Pile foundation minimum axial force (ton)
Vu = Factored shear force
β_{column} = Ratio between the width and height dimensions of the column
ϕ = Shear strength reduction factor
Vc = Concrete shear strength
α_s = Column location-dependent constant value
ρ_{min} = Minimum reinforcement ratio
k_w^1 = Stiffness constant of one pile in vertical direction
E_p = Modulus of elasticity of pile material; Young's modulus of pile
A = Area of cross-section of H-pile section
r_0 = Effective radius of one pile, equivalent radius; radius of the pile
f_w^1 = Vertical stiffness parameter of a single pile
α_A = Axil displacement interaction factor for a typical reference pile in a group
k_w^g = Stiffness constant of pile group in vertical direction
k_w^f = Stiffness constant of pile cap in vertical direction
G_s = Shear modulus of the soil on the sides of the pile

h = Depth of embedment; length of pile above ground

$\overline{S_1}$ = Frequency-independent parameter of side layer for vertical vibration

k_w = Total stiffness constant in vertical direction

L/r_0 = Slenderness ratio

k_x^1 = Spring constant of single pile in lateral direction (translation)

Ip = Moment of inertia of pile (mm^4)

f_x^1 = Horizontal (sliding) stiffness parameter of a free head pile

α_L = Lateral displacement interaction factor for a typical reference pile in a group

k_x^g = Stiffness constant of pile group in lateral direction (translation)

k_x^f = Stiffness constant of pile cap in lateral direction (translation)

$\overline{S_{x1}}$ = Frequency-independent parameter of side layer for horizontal sliding

k_x = Total stiffness constant in lateral direction (translation)

k_ϕ^1 = Stiffness constant of single pile in rocking

f_ϕ^1 = Rocking stiffness parameter of a pile

$k_{x\phi}^1$ = Cross spring stiffness of single pile

$f_{x\phi}^1$ = Cross stiffness parameter

k_ϕ^g = Stiffness constant of pile group in rocking

x_r = Distance of each pile from the C.G. (center of gravity)

z_c = Height of center of gravity of the pile cap above its base

k_ϕ^f = Stiffness constant of pile cap in rocking

$\overline{S_{\phi 1}}$ = Frequency-independent side layer parameter for torsional vibration

δ = Angle of friction between soil and pile

k_ϕ = Total stiffness constant in rocking

CHAPTER I
INTRODUCTION

1.1 Background

In a construction project, the foundation is an important element that functions as a bearer and retainer of all structural loads that are above it and transmits it to the subsoil below. Consequently, the foundation of a building must be designed so that it can support the load of the superstructure up to a certain safety limit.

As mentioned in the book Piles and Pile Foundations by Viggiani, Mandolini, & Russo (2012), pile foundations have been in use since prehistoric times and have been evolving since then. Today, pile foundations aid the same purpose, which is to make it possible to build in areas where the soil conditions are unfavorable for foundations. Piles may be used to support the foundations in buildings, machines, and offshore structures. There are two types of pile foundations, which are fixed and flexible. These piles experience static and dynamic internal forces, including axial and lateral.

According to Coduto, Kitch, & Yeung (2016), there are two types of foundations, namely shallow and deep foundations. As opposed to the shallow foundation that transfers building loads to a subsurface layer or a range of depths, a deep foundation transfers the loads to the subsoil farther down from the surface. In this research, a deep foundation is used to withhold the load of a 15-story building structure.

Deep foundation can be designed as fixed or flexible according to the soil parameters, seismic condition of the area, and the characteristics of the building. This research will compare the effect of internal forces working on fixed and flexible foundations using the same building to obtain the more suitable type of pile foundation in certain conditions.

Deepa, Mithanthaya, & Venkatesh (2021) completed a study in which the objective is to obtain a comparison result between fixed and flexible bases using an identical symmetrical building. Both models are subjected to linear static analysis and nonlinear pushover analysis, whereas the variation in displacement, base force, and fundamental period are observed. Both the linear and nonlinear analysis performed in the case of the fixed base shows lesser displacement, base force, and fundamental period compared to the flexible continuum model. However, this study is still lacking due to the limitations of the symmetrical building or regular plan. An extensive study with an irregular building plan is needed to further compare the effects of fixed and flexible foundations on structures.

Li, Escoffier, & Kotronis (2020) presented a comprehensive study on the different behavior of batter (inclined) and vertical pile foundations in terms of stiffness degradation and damping properties under dynamic loadings by conducting a series of centrifuge tests. The result of the tests showed that the presence of batter piles increases the rotational damping ratio without losing much of the rotational stiffness for rocking behavior, whereas batter piles have a more important horizontal stiffness than the vertical and energy dissipation ability for horizontal translational behavior. To implement the foundation stiffness degradation and damping curves, the study adopted the Equivalent Linear Approach (ELA). However, this approach was not able to take into account the strong strain dependence on secant modulus and damping ratio. Other than that, due to the limitation of accelerometers in the centrifuge tests, the residual displacement or rotation could not be obtained, which was also the case for the equivalent linear approach.

An attempt had also been made in a study by Chougule & Dyavanal (2015) to analyze the effect of Soil-Structure Interaction (SSI) on multi-storied buildings with various foundation systems. This study analyzed the response of multi-storied buildings subjected to seismic forces with rigid and flexible foundations subjected to seismic forces under different soil conditions. The results showed that the fundamental natural frequencies of a building increase and base shears decrease with the increase of soil stiffness and this change is found more in soft soils. Lateral deflection, story drift, and base shear values of the fixed base building were found to be lower as compared to flexible base buildings. However, after observing the performance points of all building models, it was concluded that injuries during the earthquake may still occur, despite the life-threatening risk from the structural damage being very low. This means that the execution of a retrofit may still be needed.

Based on some limitations of these previous research, the purpose of this study is to conduct a continuation of research by overcoming these limitations. Therefore, it means that this research is conducted to continue the development of studies as well as provide a deeper comprehension of the topic of internal forces working on fixed and flexible foundations. Not only that, but this study is also to design a symmetrical building with a lower risk of structural damage to improve the performance of the building without needing a retrofit or a redesign, to highlight the impact of soil conditions under earthquake load in fixed and flexible foundations, as well as to emphasize the difference of response between fixed and flexible foundations. However, for the flexible foundation, this study will only consider the spring stiffness parameters, instead of stiffness and damping due to some limitations.

Based on the description of the background above, the research title that the writer will raise in this final project is "Comparative Analysis Between Internal Forces of Fixed and Flexible Foundation Under Earthquake Load." This study suggests the importance of pile foundation properties—especially in the case of spring stiffness parameters—towards SSI. With the results obtained, this study may also be used as a reference material to re-analyze the use of fixed and flexible foundations and their effects on SSI, which contributes to the development of Soil-Structure Interaction studies.

1.2 Research Objectives

The research objectives are as follows.

1. How is the effect of the flexible foundation on the period of vibration of the structure as well as the internal forces caused by earthquake load?

2. What is the difference between internal forces in fixed and flexible pile foundations?
3. What are the effects of soil shear modulus distribution if it is assumed to be uniformly distributed along the pile length and parabolic to the stiffness interaction?

1.3 Purpose

The purposes of this final project are as follows.

1. To analyze the effect of the flexibility of pile foundation towards the natural period of vibration of a building, as well as its effect towards seismic demands imposed by ground motions.
2. To compare the internal forces acting upon fixed and flexible pile foundations in medium soil.
3. To reveal the effects of soil shear modulus distribution when assumed to be uniformly distributed along the pile length and parabolic to the stiffness interaction on the structure design.

1.4 Advantage

The advantages of this final project are as follows.

1. Code SNI 1726:2019 has allowed the use of flexible foundations in building designs, encouraging structural engineers to consider the effect of flexible foundations when used in a design.
2. To be used as reference material for readers to re-analyze the use of fixed and flexible foundations.
3. To increase knowledge for readers about the stiffness of pile foundations.

1.5 Limitation

This research requires boundaries to be directed and focused, hence the following limitations are made.

1. The types of foundations observed are limited to fixed and flexible piles with the same superstructure design.

2. Structural analysis is limited to static analysis, as it does not include Time History Analysis (THA).
3. The superstructure design is a 15-story building using reinforced concrete without a structural shear wall.
4. The designed structure is located in Pleret, Imogiri, Bantul, Yogyakarta with medium soil according to SNI 1726:2019.
5. The loading design refers to SNI 1727:2020.
6. Structural member design refers to SNI 2847:2019 concerning Structural Concrete Requirements for Buildings.
7. Slab design analysis refers to Indonesian Reinforced Concrete Regulations (PBI 1971).
8. Building structural design uses the program of ETABS V 18.1.1.
9. Seismic provisions refer to Federal Emergency Management Agency (FEMA) regulations P-750 and P-751.
10. Soil-Structure Interaction (SSI) analysis refers to the regulations of the Federal Emergency Management Agency (FEMA) P-2091.
11. The calculation of spring stiffness refers to the book Pile Foundations in Engineering Practice by Prakash & Sharma (1990).
12. The quality of the material in the structure has the following characteristics.
    a. Concrete quality in structural members, f'c = 35 MPa
    b. Quality of steel reinforcement with diameter > 12 mm, fy = 400 MPa
    c. Quality of steel reinforcement with diameter < 12 mm, fy = 360 MPa
13. The structural design includes the following components.
    a. Main beam design
    b. Secondary beam design
    c. Column design
    d. Floor plate design
    e. Roof plate design
    f. Stairs design
    g. Foundation design

14. The superstructure is designed with some voids around the center of each story to accommodate four units of lifts/elevators, however the weight of each unit is not considered in the analysis of total building weight.
15. Soil N-SPT data for foundation design is obtained from the Soil Mechanics laboratory of the Faculty of Civil Engineering and Planning, Universitas Islam Indonesia.
16. Flexible foundation parameters are limited to spring stiffness only, not taking into account the dashpot damping due to the limited time of design.
17. As the foundation spring stiffness is constant, the results are limited to the elastic response.
18. The internal forces analyzed to compare fixed and flexible foundations are shear force, bending moment, drift ratio, joint rotation, as well as horizontal joint displacement.
19. The soil profile considered in the flexible foundation design for the comparison analysis between fixed and flexible foundations is only the homogeneous soil profile due to the limited time of design.
20. The results of spring stiffness analysis with parabolic soil profile are only briefly compared with the homogeneous soil profile instead of an extensive analysis.

CHAPTER II
LITERATURE REVIEW

2.1 General

Piles are widely known as part of a structural foundation that is exposed to dynamic loads such as wind or earthquakes. In buildings, piles facilitate the transfer of loads to deeper depths, as it is needed when soils near the ground surface are of poor quality. The introduction of the pile stiffens the system, and both the natural frequency and the amplitude of motion are affected. In all vibration problems, resonance needs to be avoided. Therefore, the natural frequency of the soil pile system needs to be evaluated. Nevertheless, the dynamic behavior of piles is yet to be completely understood due to the complexity of soil-pile interaction. The dynamic response of a structure supported by piles can be predicted if the dynamic stiffness and damping generated by soil-pile interaction can be defined (Novak, 1974).

Even though single piles are still frequently used, piles are generally used in groups. According to the book Pile Foundations in Engineering Practice published by Prakash & Sharma (1990), the stiffness and damping of the pile group need to be evaluated in light of the group action. It is incorrect to assume that the group stiffness and damping are the simple sums of the stiffness and damping of individual piles. The extent of group action depends on the ratio of the spacing to the pile diameter. A smaller spacing will result in a larger group action and contrariwise.

Internal force analysis of pile foundation has been studied, for both single and group piles. These piles may experience dynamic and static loads, consisting of vertical (axial) and lateral vibrations. In this research, the internal forces of pile foundations will be elaborated on and compared between fixed and flexible foundations. Further descriptions of previous studies regarding this topic and their limitations are presented in this chapter.

2.2 Symmetric Building with Fixed and Flexible Base

Typically, structural systems transfer the building load through a series of elements, namely the structural foundation, to the ground. Each joint is designed to transfer or support a specific type of load or loading condition. To examine a structure, it is initially necessary to be certain about the forces that can be resisted and transferred at each level of support throughout the structure. Deepa, Mithanthaya, & Venkatesh (2021) supported this concept in their study and proposed that the type of support connection regulates the type of load that the support can withstand. The support type also has a valid influence on the load-bearing capacity of each element, as well as the system. Moreover, it was also studied that inertia developed in a vibrating structure gives rise to base shear, moment, and torsion. These forces generate displacements and rotations at the soil-foundation interface. The study aims to obtain a comparison result between fixed and flexible bases using an identical symmetrical building. Both models are subjected to linear static analysis and nonlinear pushover analysis, whereas the variation in displacement, base force, and fundamental period are observed.

A 3D model of a 25 m x 25 m plan and 30 m height was used in the building design in SAP 2000. The raft foundation was modelled at the base of the 3D building with an area of 27 m x 27 m with 0.75 m thickness. The parameters selected for modeling the soil were shear modulus and Poisson's ratio which was calculated as per standards ASCE 41-13. The analysis was performed on a 3D, 10-story building using SAP 2000 V.19.2.1 software, which fulfilled the conditions for both SSI as well as nonlinear analysis. The two models considered were regular buildings with fixed bases and regular buildings resting on 3D soil flexible base (continuum model).

The linear analysis performed in the case of the fixed base shows lesser displacement, base force, and fundamental period compared to the flexible continuum model. It was possibly caused by the use of a continuum soil model that was considered a flexible base. The soil properties incorporated increased the displacement, base force, and fundamental period. Accordingly, the nonlinear analysis performed in the case of the fixed base also showed lesser displacement, base force, and fundamental period compared to the flexible continuum model. This was because nonlinear pushover analysis considered only the building results regardless of the soil condition. Another possible cause was the use of a symmetrical building or regular plan which was considered without a soil base. Overall observation noticed that nonlinear analysis in the case of the regular symmetrical building showed better results with a fixed base compared to the flexible base. The results also indicated that it is necessary to perform analysis on irregular buildings with a fixed base to study the changes in the behavior of the building when subjected to nonlinear analysis. Consequently, this indicates that this study is still lacking due to the limitations of the symmetrical building or regular plan. An extensive study with an irregular building plan is needed to further compare the effects of fixed and flexible foundations on structures.

2.3 Spring Stiffness of Pile Foundations

Currently, studies for stiffness degradation and energy dissipation properties for deep foundations are rare, and existing studies mainly focused on shallow foundations. Li, Escoffier, & Kotronis (2020) presented a comprehensive study on the different behavior of batter (inclined) and vertical pile foundations in terms of stiffness degradation and damping properties under dynamic loadings. Their study outline includes the different behaviors in terms of stiffness degradation and damping properties of batter and vertical pile foundations, which are emphasized by a sequence of centrifuge tests. Subsequently, based on the experimental outcomes, stiffness degradation and damping curves are proposed for both batter and vertical pile foundations. To conclude, numerical validation using an equivalent linear approach with an iterative process is utilized to validate the proposed curves. The comparison of numerical and experimental results shows good agreement. This study also incorporated the importance of SSI in foundation design along with the analysis of pile foundation stiffness degradation and damping properties under dynamic loadings.

In the study, an experimental campaign was carried out to investigate the rocking and lateral translation behavior of batters and vertical foundations. Three types of single-degree-of-freedom superstructures were used in the tests, i.e., short, medium-tall, and tall superstructures. These superstructures were designed to have the same fixed base frequency; the same top mass weight; and the same total weight of the whole foundation-superstructure system. In the experiments, the responses of the foundation and the superstructure are recorded by sets of accelerometers. Meanwhile, in the dynamic centrifuge test, the soil-pile-superstructure system is loaded at the base which is close to the real loading case of ground motion. Both kinematic and inertial interaction can be included in dynamic centrifuge tests. Under dynamic loading, pile foundations move horizontally combined with rocking movements. The rocking and translational behavior of the foundations can be significantly influenced by the presence of batter piles. The result of the tests showed that the presence of batter piles increases the rotational damping ratio without losing much of the rotational stiffness for rocking behavior, whereas batter piles have a more important horizontal stiffness than the vertical and energy dissipation ability for horizontal translational behavior. Furthermore, the results from different dynamic loadings have very similar tendencies. This shows that the behavior of foundations is not influenced by superstructures.

To implement the foundation stiffness degradation and damping curves, the study adopted the Equivalent Linear Approach (ELA). From the results, it was proved that for a batter (or vertical) foundation with a short (or medium-tall and tall) superstructure, the numerical model has a good performance in replicating the dynamic response of foundations under various dynamic loadings. However, some limitations of the numerical simulations should be acknowledged. Firstly, the ELA adopted in this study was not able to take into account the strong strain dependence on the secant modulus and damping ratio. For large deformation under strong ground motions, using constant secant stiffness and damping ratio may not yield good results. Secondly, due to the limitation of accelerometers in the centrifuge tests, the residual displacement or rotation could not be obtained, which was also the case for the equivalent linear approach. In this study, the good agreement of the comparisons only referred to dynamic displacements and rotations. However, despite these limitations, the proposed model may be utilized for the preliminary evaluation of the nonlinear SSI of pile foundations.

2.4 Seismic SSI of Buildings with Fixed and Flexible Foundation

The effect of Soil-Structure Interaction (SSI) cannot be disregarded in the design process of low-rise buildings laying on shallow foundations as it may lead to an unsafe seismic design. When a structure is subjected to earthquake excitation, the interaction between the foundation and soil occurs, thus changing the motion of the ground. This indicates that the movement of the whole ground structure system is influenced by the soil as well as the structure type. An attempt had been made in a study by Chougule & Dyavanal (2015) to analyze the effect of SSI on multi-storied buildings with various foundation systems. This study also analyzed the response of multi-storied buildings subjected to seismic forces with rigid and flexible foundations subjected to seismic forces under different soil conditions, such as hard, medium, and soft soils. The study chose a conventional G+6 story building resting on different soils. The influence of SSI is compared with the results obtained when the structure is assumed to be fixed at the base.

This study proposed the use of pushover analysis in assessing seismic SSI. Pushover analysis is a static, nonlinear procedure in which the magnitude of the structural load is increased gradually according to a certain predetermined pattern. Having developed modelling procedures, acceptance criteria, and analysis procedures, the ATC-40 and FEMA-273 documents were used as a source of provisions for pushover analysis. These documents define force-deformation criteria for hinges used in this analysis. The typical building used in the design measured 15 m x 15 m each bay of 3 m width in a G+6 story building with infill bricks and bare framing modelled in ETABS software. The basement level is maintained at 4.8 m and a typical level of 3.6 m is maintained for the rest of the stories.

The results showed that the fundamental natural frequencies of a building increase and base shears decrease with the increase of soil stiffness and this change is found more in soft soils. Lateral deflection, story drift, and base shear values of the fixed base building were found to be lower as compared to flexible base buildings. This shows that a suitable foundation system considering the effect of soil stiffness must be adopted while designing building frames for seismic forces. After observing the performance points of all building models, it was concluded that injuries during the earthquake may still occur, despite the life-threatening risk from the structural damage being low. This corresponds to the limitation of the study, which is to design a more suitable building by executing a retrofit that will accordingly improve the performance of the building so that it can be increased to the life safety range.

2.5 Summary of Research

The summary of the previously mentioned research regarding fixed and flexible foundations can be seen in the following table.

Table 2.1 Summary of Research

Aspect	Previous Study			Present Study
	Deepa, Mithanthaya, & Venkatesh (2021)	Li, Escoffier, & Kotronis (2020)	Chougule & Dyavanal (2015)	Syafira Nurulita (2023)
Title of Study	Comparison of Symmetric Building with Fixed Base and Flexible Base Continuum Model in SAP 2000 V.19.2.1	Study on the Stiffness Degradation and Damping of Pile	Seismic Soil Structure Interaction of Buildings with Rigid and Flexible Foundation	Comparative Analysis Between Internal Forces of Fixed and Flexible Foundation Under Earthquake Load
Purpose of Study	To study the variation in displacement, base force, and the fundamental period in a symmetrical or regular building modeled by the continuum method with fixed and flexible base	To present a comprehensive study on the different behavior of batter and vertical pile foundations in terms of stiffness degradation and damping properties under dynamic loadings	To study the effect of soil-structure interaction on multi-storied buildings with various foundation systems	To compare the internal forces acting upon fixed and flexible pile foundations in medium soil
Structural Model	A 10-story building with a fixed and flexible base	Short, medium-tall, and tall superstructures with batter and vertical pile foundations	G+6-story building with a rigid and flexible foundation	A 15-story building with a fixed and flexible foundation

Loading Code	ASCE41-13	ASCE	ATC-40 FEMA-273 IS:1893 IS:456-2000	SNI 1727:2020
Method of Analysis	Building modelled using SAP200 V 19.2.1 and subjected to linear static analysis and nonlinear static pushover analysis	Building modelled as short, medium-tall, and tall superstructures and the piles are subjected to a series of centrifuge tests with the numerical validation using an Equivalent Linear Approach (ELA)	Building modelled using ETABS V 9.7 and subjected to pushover analysis as a static and nonlinear procedure	Building modelled using ETABS V 18.1.1 where the foundation is subjected to a substructural approach
Result	Both analyzes performed in the case of the fixed base show lesser displacement, base force, and fundamental period compared to the flexible continuum model	For a batter (or vertical) foundation with a short (or medium-tall and tall) superstructure, the numerical model has a good performance in replicating the dynamic response of foundations under various dynamic loadings	Lateral deflection, story drift, and base shear values of the fixed base building were found to be lower as compared to the flexible base building	
The Difference with Present Study	The present study models a higher building with 15 stories using ETABS V 18.1.1	The present study models a tall superstructure and analyzes the building using Equivalent Lateral Force (ELF) and Response Spectrum (RS) methods instead of performing a series of centrifuge tests on the piles	The present study models a higher building with 15 stories using ETABS V 18.1.1 and analyzes the building using Equivalent Lateral Force (ELF) and Response Spectrum (RS) methods	

2.6 Originality of Research

Compared with previous studies, there are several differences with this final project. The study by Deepa, Mithanthaya, & Venkatesh (2021) analyzes a 10-story building modelled using SAP V 19.2.1 with a fixed and flexible base, while the present study analyzes a higher building with 15 stories modelled using ETABS V 18.1.1. The loading code used is also different. For the comparison with the study by Li, Escoffier, & Kotronis (2020), the superstructures analyzed are short, medium-tall, and tall buildings with batter and vertical piles in which they are subjected to a series of centrifuge tests, while the numerical validation uses Equivalent Linear Approach (ELA). In the present study, however, the building modelled is a tall superstructure that is analyzed by Equivalent Lateral Force (ELF) and Response Spectrum (RS) methods. For the study by Chougule & Dyavanal (2015), the building has G+6 stories modelled using ETABS V 9.7 and subjected to pushover analysis as a static and nonlinear procedure, while the present study models a higher building with 15 stories using ETABS V 18.1.1.

The previous studies discussed a similar topic to the present study. However, this study will also highlight the effect of the flexibility of pile foundation towards the natural period of vibration of a building, as well as its effect on seismic demands imposed by ground motions. Other than that, this study will also compare the internal forces acting upon fixed and flexible pile foundations in medium soil as well as reveal the effects of soil shear modulus distribution when assumed to be uniformly distributed along the pile length and parabolic to stiffness and damping interaction on the structure design. These are the purposes of the present study, which ensure this study's originality and avoid plagiarism with the previous studies.

CHAPTER III
THEORETICAL BASIS

3.1 Pile Foundation

In structural analysis, interactions between soil-foundation and structure are important. Soil-structure interaction (SSI) can make a big difference in how a building behaves during earthquake vibration and how it should be designed. The foundation of a building serves an important role in influencing how it behaves when it receives earthquake forces.

According to ASCE/SEI 7-16 and ASCE/SEI 41-17, motion on the ground surface without any structures and foundations is called free-field motion. Meanwhile, the kinematic interaction is a source of modification of the free-field motion into the foundation input motion applied to the ends of the horizontal foundation springs. It differs from actual foundation motion due to the inertial response of the structure and the deflection produced by the response in the foundation springs. This response is known as inertial interaction.

This research will analyze the building behavior and SSI using a deep foundation, in this case, the pile foundation. It is a collection or group of piles used as a structural element that connects the building to the ground and transfers loads to the subsoil. The group of piles is bound by a thick concrete mat that rests on a pile of concrete that has been driven into soft or unstable soil to provide a suitable stable foundation. Internal forces analysis (shear force, flexural moment, drift ratio, joint rotation, horizontal joint displacement, etc.) used in the structural design should always consider whether structural safety is ensured. The internal forces working under dynamic loads will further be compared between fixed and flexible foundations. The calculation of this analysis must also be precise.

3.2 Fixed and Flexible Foundation

The interface between the foundation and the soil supports is determined by the most important parameter, the bearing pressure. It is the contact force per unit area along the bottom of the footing. The stiffness of footing, the compressibility of soil, and the type of loading all affect the bearing pressure distribution of a foundation. As a result, the bearing or contact pressure of fixed (rigid) and flexible foundations differ.

In a fixed (rigid) foundation, the soil beneath the footing generally experiences nonlinear pressure distribution. Meanwhile, in a flexible foundation, the soil beneath the footing generally experiences linear pressure distribution. The conditions may differ according to the type of soil, i.e., cohesive or cohesionless soil.

As the name suggests, a fixed (rigid) foundation settles as a rigid element, whereas a flexible foundation is considered to have some degree of flexibility. A fixed (rigid) foundation does not experience bending (curvature) along its length or width even if it is subjected to concentrated loading. On the other hand, a flexible foundation will bend if it experiences partial pressure or concentrated load due to its flexibility.

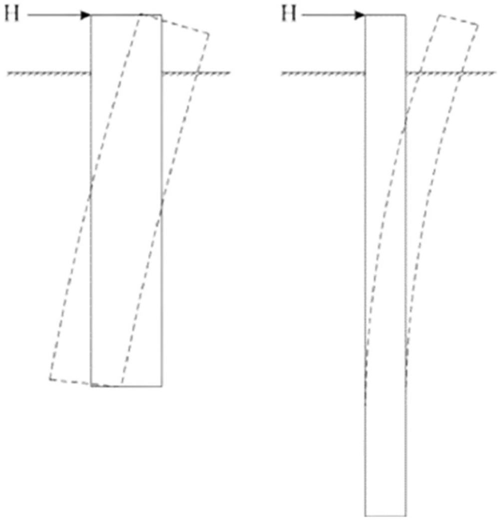


Figure 3.1 Fixed Foundation (left) and Flexible Foundation (right) Submitted to Lateral Load

(Source: Jenck, Obaei, Emeriault, & Dano, 2021)

To differentiate from a fixed foundation, the principles of a flexible foundation must be determined. Flexible foundations are principally calculated using the same equations as fixed (rigid) foundations. However, several data inputs differ. These differences include:

1. The length of the pile is greater than the fixed foundation.
2. Slenderness ratio (L/r_0) is greater than fixed foundation.
3. Pile stiffness (K_R) is < 0.01.
4. The α_L parameters in Figure 3.20 are obtained using the dotted lines.

3.3 Dynamic Loads

The conventional building design is most often carried out based on the assumption that the building essentially stays on rigid ground. This assumption is not entirely correct but is often considered acceptable and conservative. The flexibility of the foundation system, including the structural components of the foundation and soil supports, can have a significant effect on the dynamic properties of a building and its overall response.

Soil flexibility in the analytical model of a building is generally calculated by modeling the connection of structural elements to fixed supports with spring elements. Vertical and rotational springs affect a structure that sways essentially due to elastic vertical compression of the soil—either through vertical movement of the ends of a frame, or rotation at the base of the wall or core. This behavior can have dramatic effects on the building's fundamental period and displaced forms.

The horizontal spring models the displacement of the foundation relative to the displacement of the free-field soil or the resistance of the soil to a basement wall or other vertical surface. Spring stiffness is limited by friction resistance and passive pressure.

The shear modulus (G) of soil is one of the main dynamic properties that affect soil behavior under vibrational loading. Laboratory tests have shown that the soil shear modulus of soil stiffness is affected by several factors, such as cyclic strain amplitude, void ratio, mean principal effective stress, plasticity index, over-consolidation ratio, and a few loading cycles. The maximum shear modulus (G_{max})

of the soil can also be estimated from the in-situ test results. Several empirical relationships between G_{max} and various in-situ parameters have been developed by performing standard penetration tests (SPT) and cone penetration tests (CPT).

3.4 Dimension Estimation

Before analyzing the fixed and flexible foundations and their internal forces, the building must be designed beforehand. The first step is to estimate the dimensions of the structural members, such as the beams, columns, floor and roof plates, as well as stairs.

3.4.1 Main Beam Dimension Estimation

Based on SNI 2847:2019 as listed in Table 9.3.1.1, the minimum height of a non-prestressed beam with a simple attachment condition is 1/16 of the span length of the beam. However, the formula used to calculate the main beam's height is modified to obtain a safer beam dimension. The height of the main beam is determined according to the following equation based on the span length of the beam.

$$H_{\text{main beam}} = \frac{1}{10} \cdot L_{\text{main beam}} \tag{3.1}$$

with:

$H_{\text{main beam}}$ = height dimension of main beam

$L_{\text{main beam}}$ = span length of main beam

Meanwhile, the width (B) of the beam is estimated as half the height (H).

3.4.2 Secondary Beam Dimension Estimation

Like the calculation of the main beam, the minimum height of the secondary beam also uses the formula of 1/16 of the span length of the beam. However, the formula used to calculate the secondary beam's height is modified to obtain a safer beam dimension. The height of the secondary beam is determined according to the following equation based on the span length of the beam.

$$H_{\text{secondary beam}} = \frac{1}{12} \cdot L_{\text{secondary beam}} \tag{3.2}$$

with:

$H_{\text{secondary beam}}$ = height dimension of secondary beam

$L_{\text{secondary beam}}$ = span length of secondary beam

Meanwhile, the width (B) of the beam is estimated as half the height (H).

3.4.3 Column Dimension Estimation

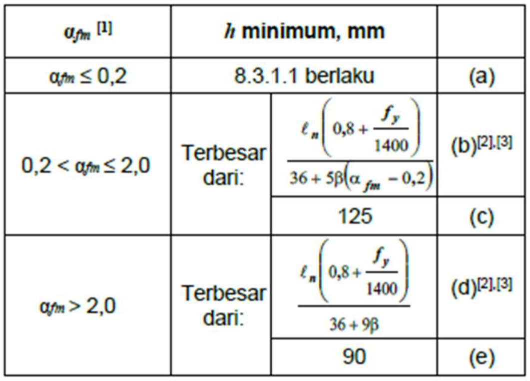
Based on SNI 2847:2019 Article R10.3.1 regarding column dimensional limitations, an explicit minimum size on a column is not specified. The column dimensions are estimated according to the size of the building.

3.4.4 Floor Plate Dimension Estimation

According to SNI 2847:2019, if the value of Ly/Lx is less than 2, the slab is considered two-way. On the other hand, if it is equal to or exceeds the value of 2, the slab is considered one-way.

Based on SNI 2847:2019, the minimum thickness (h) requirement of a two-way non-prestressed slab with beams between supports on all sides is as follows.

Table 3.1 Minimum Thickness of Two-Way Non-Prestressed Slab with Beams between Supports on All Sides



α_{fm} [1]	*h* minimum, mm		
$\alpha_{fm} \le 0{,}2$	8.3.1.1 berlaku		(a)
$0{,}2 < \alpha_{fm} \le 2{,}0$	Terbesar dari:	$\frac{\ell_n\left(0{,}8+\frac{f_y}{1400}\right)}{36+5\beta\left(\alpha_{fm}-0{,}2\right)}$	(b)[2],[3]
		125	(c)
$\alpha_{fm} > 2{,}0$	Terbesar dari:	$\frac{\ell_n\left(0{,}8+\frac{f_y}{1400}\right)}{36+9\beta}$	(d)[2],[3]
		90	(e)

(Source: SNI 2847:2019 Table 8.3.1.2, p.135)

The ratio of the bending stiffness of the beam section to the bending stiffness of the plate width limited laterally by the center line of the adjacent panel (if any) on each side of the beam (α_f) is determined as follows.

$$\alpha_f = \frac{E_{cb} \cdot I_b}{E_{cs} \cdot I_s} \qquad (3.3)$$

Furthermore, the average value of α_f for all beams at the edge of the panel is shown as follows.

$$\alpha_{fm} = (\alpha_{f1}+\alpha_{f2}+\alpha_{f3}+\alpha_{f4})/4 \qquad (3.4)$$

with:

Ecb = elastic modulus of the concrete beam (MPa)

Ecs = elastic modulus of the concrete slab (MPa)

Ib = moment of inertia of the beam gross cross-section about the central axis (mm^4)

Is = moment of inertia of the slab gross cross-section about the central axis (mm^4)

3.4.5 Roof Plate Dimension Estimation

The estimated dimension calculation process for the roof plate/slab including minimum thickness is the same as the floor plate/slab.

3.4.6 Stairs Dimension Estimation

The stair geometric design includes some general requirements, such as:

1. The width of the stairs and landing meets the needs.
2. The length of the stairs is sufficient so that it can provide proportional and safe *antrede* (a) and *optrede* (s).
3. A strong and safe handrail.
4. Meets structural requirements.

Meanwhile, the requirement for step measurements is as follows.

$$59 \text{ cm} \leq (2s + a) \text{ cm} \leq 65 \text{ cm} \qquad (3.5)$$

The illustration of *antrede* (a) and *optrede* (s) of stairs is shown in the following figure.

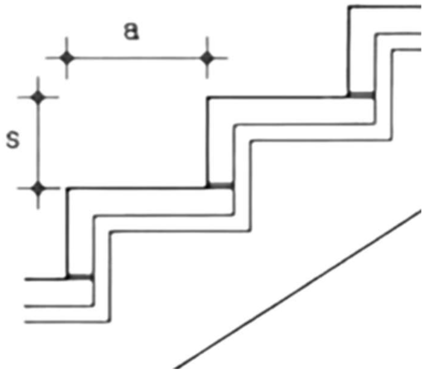


Figure 3.2 Illustration of *Antrede* (a) and *Optrede* (s) of Stairs

Moreover, the dimension requirements for stair width, *antrede* (a) width, and *optrede* (s) height are as follows.

1. Stair width: Minimum 80 cm, dependent on the building area and number of residents.
2. *Antrede* (a) width: Maximum 19 cm (general), or 21 cm (residence).
3. *Optrede* (s) height: Minimum 26 cm.

After obtaining the height and width of the stairs, the number of stairs (n) is also calculated. The equation used is as follows.

$$n = (H_{story}/s) - 1 \quad (3.6)$$

Meanwhile, the equation used to calculate the stairs tilt angle, α (in degrees) is as follows.

$$\alpha = \arctan (s/a) \quad (3.7)$$

Finally, the equation used to calculate the equivalent thickness of stairs (in cm) is as follows.

$$t1 = (1/2)\ s \times \cos \alpha \quad (3.8)$$

3.5 Seismic Design of the Structural Model

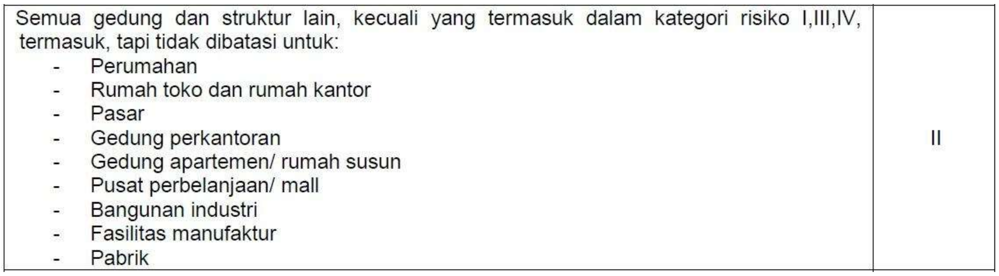
The structural seismic design considerations used in this study are elaborated in the following subchapters.

3.5.1 Data Determination

1. Classification of risk category and importance factor, Ie

   The requirement for building risk category according to SNI 1726:2019 is as follows.

Table 3.2 Building Types in Risk Category II

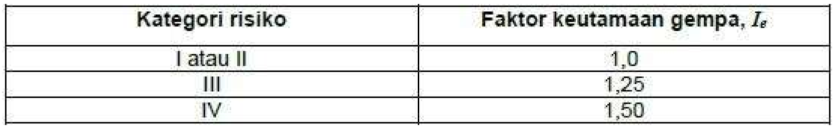


Semua gedung dan struktur lain, kecuali yang termasuk dalam kategori risiko I,III,IV, termasuk, tapi tidak dibatasi untuk: - Perumahan - Rumah toko dan rumah kantor - Pasar - Gedung perkantoran - Gedung apartemen/ rumah susun - Pusat perbelanjaan/ mall - Bangunan industri - Fasilitas manufaktur - Pabrik	II

(Source: SNI 1726:2019 Table 3, p.24)

Meanwhile, the requirement for importance factor, Ie according to SNI 1726:2019 is as follows.

Table 3.3 Importance Factor, Ie



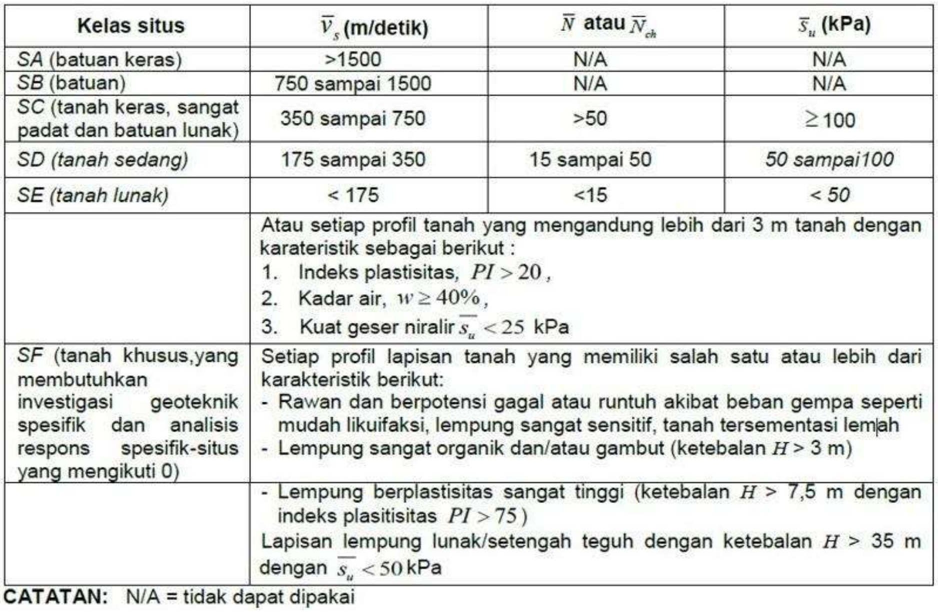
Kategori risiko	Faktor keutamaan gempa, I_e
I atau II	1,0
III	1,25
IV	1,50

(Source: SNI 1726:2019 Table 4, p.25)

2. Classification of soil site

The soil site class classification according to SNI 1726:2019 is as follows.

Table 3.4 Soil Site Classification



Kelas situs	$\bar{v}_s$ (m/detik)	$\bar{N}$ atau $\bar{N}_{ch}$	$\bar{s}_u$ (kPa)
SA (batuan keras)	>1500	N/A	N/A
SB (batuan)	750 sampai 1500	N/A	N/A
SC (tanah keras, sangat padat dan batuan lunak)	350 sampai 750	>50	$\geq$100
SD (tanah sedang)	175 sampai 350	15 sampai 50	*50 sampai100*
SE (tanah lunak)	< 175	<15	*< 50*
	Atau setiap profil tanah yang mengandung lebih dari 3 m tanah dengan karateristik sebagai berikut : 1. Indeks plastisitas, $PI > 20$, 2. Kadar air, $w \geq 40\%$, 3. Kuat geser niralir $\bar{s}_u < 25$ kPa		
SF (tanah khusus,yang membutuhkan investigasi geoteknik spesifik dan analisis respons spesifik-situs yang mengikuti 0)	Setiap profil lapisan tanah yang memiliki salah satu atau lebih dari karakteristik berikut: - Rawan dan berpotensi gagal atau runtuh akibat beban gempa seperti mudah likuifaksi, lempung sangat sensitif, tanah tersementasi lemah - Lempung sangat organik dan/atau gambut (ketebalan H > 3 m)		
	- Lempung berplastisitas sangat tinggi (ketebalan H > 7,5 m dengan indeks plasitisitas $PI > 75$) Lapisan lempung lunak/setengah teguh dengan ketebalan H > 35 m dengan $\bar{s}_u < 50$ kPa		

CATATAN: N/A = tidak dapat dipakai

(Source: SNI 1726:2019 Table 5, p.29)

3. Ground motion parameters, Ss and S1

The Ss parameter shows the risk-targeted maximum considered earthquake ground motion (MCE_R) for the Indonesian region for the response spectrum of short periods (5% critical damping). The Ss value is determined from the classification map according to SNI 1726:2019, which can be seen as follows.

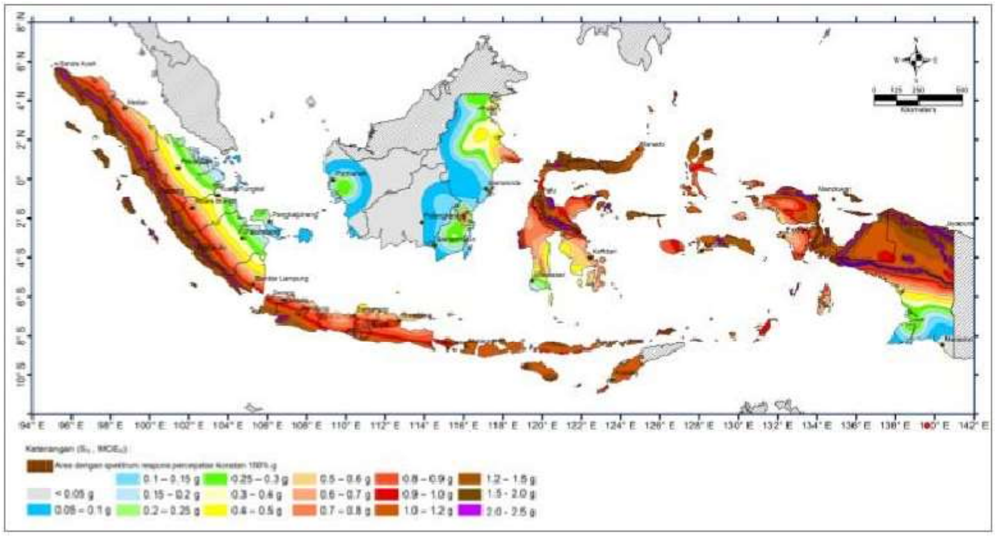


Figure 3.3 Ground Motion Parameter, Ss

(Source: SNI 1726:2019 Figure 15, p.233)

The S1 parameter shows the risk-targeted maximum considered earthquake ground motion (MCE_R) for the Indonesian region for the response spectrum of 1 second period (5% critical damping). The S1 value is determined from the classification map according to SNI 1726:2019, which can be seen as follows.

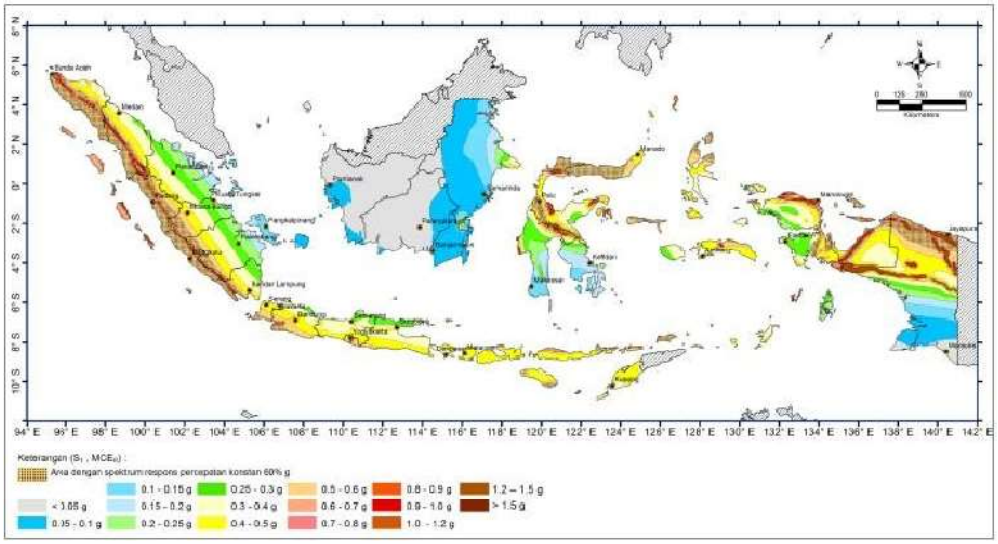


Figure 3.4 Ground Motion Parameter, S1

(Source: SNI 1726:2019 Figure 16, p.234)

4. Classification of site coefficient, Fa and Fv

Based on the following requirement by SNI 1726:2019, the value of site coefficient Fa is determined according to the following table.

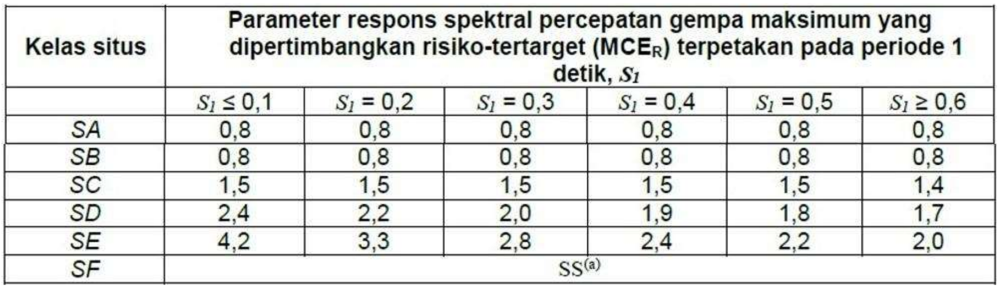
Table 3.5 Site Coefficient, Fa

Kelas situs	**Parameter respons spektral percepatan gempa maksimum yang dipertimbangkan risiko-tertarget (MCE$_R$) terpetakan pada periode pendek, T = 0,2 detik, S_s**					
	$S_s \le 0,25$	$S_s = 0,5$	$S_s = 0,75$	$S_s = 1,0$	$S_s = 1,25$	$S_s \ge 1,5$
SA	0,8	0,8	0,8	0,8	0,8	0,8
SB	0,9	0,9	0,9	0,9	0,9	0,9
SC	1,3	1,3	1,2	1,2	1,2	1,2
SD	1,6	1,4	1,2	1,1	1,0	1,0
SE	2,4	1,7	1,3	1,1	0,9	0,8
SF	SS$^{(a)}$					

(Source: SNI 1726:2019 Table 6, p.34)

Based on the following requirement by SNI 1726:2019, the value of site coefficient Fv is determined according to the following table.

Table 3.6 Site Coefficient, Fv



Kelas situs	**Parameter respons spektral percepatan gempa maksimum yang dipertimbangkan risiko-tertarget (MCE$_R$) terpetakan pada periode 1 detik, S_1**					
	$S_1 \le 0,1$	$S_1 = 0,2$	$S_1 = 0,3$	$S_1 = 0,4$	$S_1 = 0,5$	$S_1 \ge 0,6$
SA	0,8	0,8	0,8	0,8	0,8	0,8
SB	0,8	0,8	0,8	0,8	0,8	0,8
SC	1,5	1,5	1,5	1,5	1,5	1,4
SD	2,4	2,2	2,0	1,9	1,8	1,7
SE	4,2	3,3	2,8	2,4	2,2	2,0
SF	SS$^{(a)}$					

(Source: SNI 1726:2019 Table 7, p.34)

5. Response spectrum graph parameters

The parameters of the response spectrum graph are calculated using the following equations.

$$\text{SMS} = \text{Fa}\times\text{Ss} \quad (3.9)$$

$$\text{SM1} = \text{Fv}\times\text{S1} \quad (3.10)$$

$$\text{SDS} = \frac{2}{3}\times\text{SMS} \quad (3.11)$$

$$\text{SD1} = \frac{2}{3}\times\text{SM1} \quad (3.12)$$

$$T_0 = 0.2\times(\text{SD1}/\text{SDS}) \quad (3.13)$$

$$T_S = 1\times(\text{SD1}/\text{SDS}) \quad (3.14)$$

Meanwhile, the value of T_L or long-period transition can be determined from the following classification map.

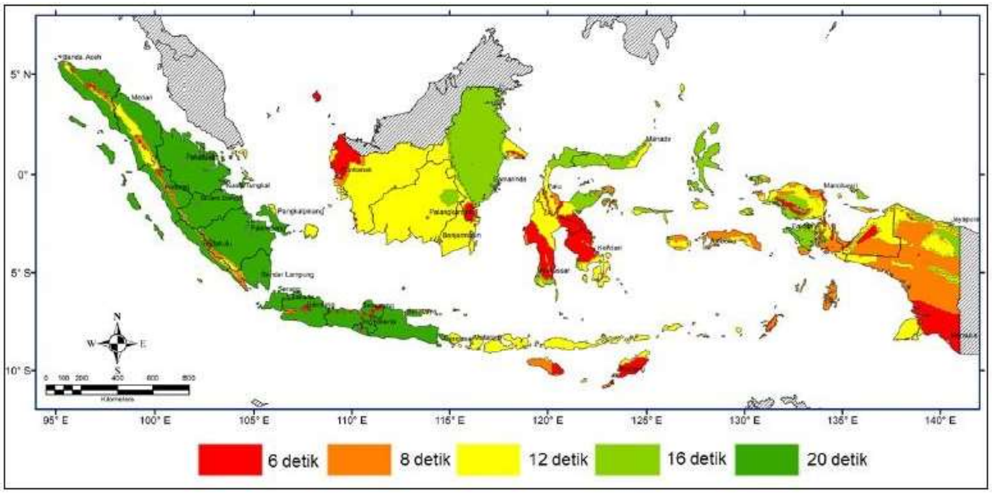


Figure 3.5 Map of Long Period Transition, T_L

(Source: SNI 1726:2019 Figure 20, p.238)

6. Seismic design category, based on SDS and SD1

Based on the following requirement by SNI 1726:2019, the seismic design category of the building based on SDS is determined according to the following table.

Table 3.7 Seismic Design Category Based on SDS

Nilai S_{DS}	Kategori risiko	
	I atau II atau III	IV
$S_{DS} < 0,167$	A	A
$0,167 \leq S_{DS} < 0,33$	B	C
$0,33 \leq S_{DS} < 0,50$	C	D
$0,50 \leq S_{DS}$	D	D

(Source: SNI 1726:2019 Table 8, p.37)

Based on the following requirement by SNI 1726:2019, the seismic design category of the building based on SD1 is determined according to the following table.

Table 3.8 Seismic Design Category Based on SD1

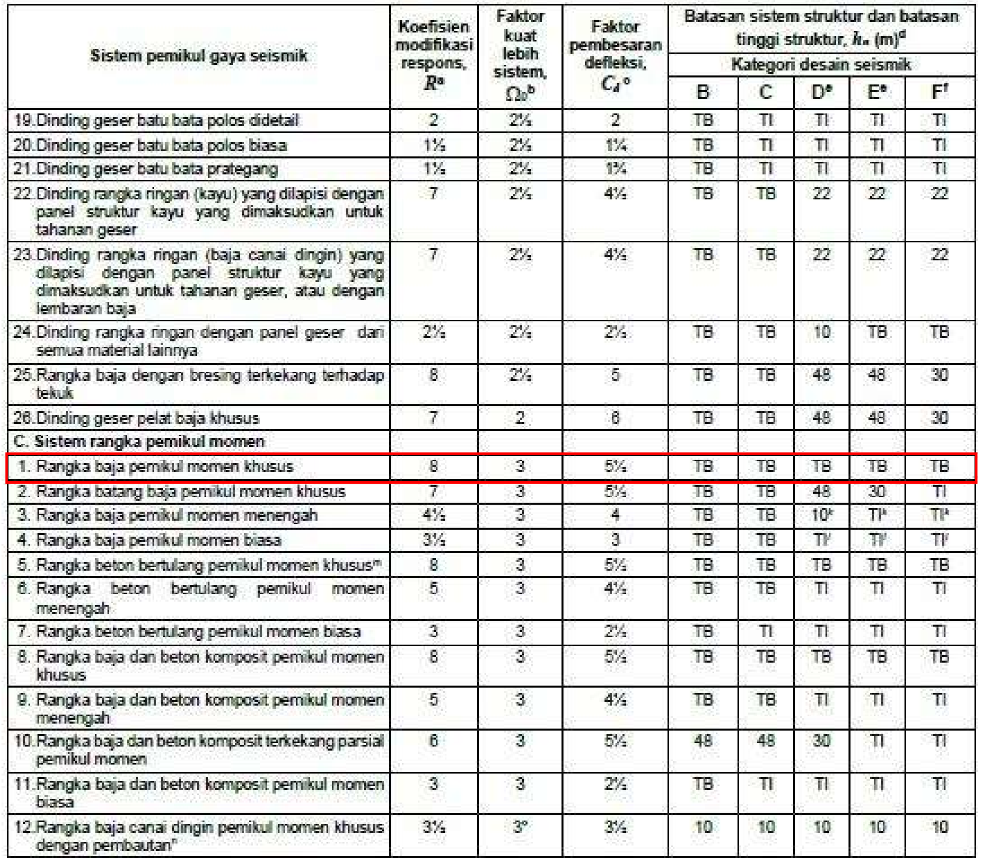
Nilai S_{D1}	Kategori risiko	
	I atau II atau III	IV
$S_{D1} < 0,067$	A	A
$0,067 \le S_{D1} < 0,133$	B	C
$0,133 \le S_{D1} < 0,20$	C	D
$0,20 \le S_{D1}$	D	D

(Source: SNI 1726:2019 Table 9, p.37)

7. Determination of R, Ω, and Cd values

   The R, Ω, and Cd values are determined according to the seismic force resisting system. The values can be determined from the following table according to SNI 1726:2019.

Table 3.9 R, Ω, and Cd Factors for Seismic Force Resisting Systems



Sistem pemikul gaya seismik	Koefisien modifikasi respons, R^a	Faktor kuat lebih sistem, $\Omega_0{}^b$	Faktor pembesaran defleksi, $C_d{}^c$	Batasan sistem struktur dan batasan tinggi struktur, h_n (m)^d				
				Kategori desain seismik				
				B	C	D^e	E^e	F^f
19. Dinding geser batu bata polos didetail	2	2½	2	TB	TI	TI	TI	TI
20. Dinding geser batu bata polos biasa	1½	2½	1¼	TB	TI	TI	TI	TI
21. Dinding geser batu bata prategang	1½	2½	1¾	TB	TI	TI	TI	TI
22. Dinding rangka ringan (kayu) yang dilapisi dengan panel struktur kayu yang dimaksudkan untuk tahanan geser	7	2½	4½	TB	TB	22	22	22
23. Dinding rangka ringan (baja canai dingin) yang dilapisi dengan panel struktur kayu yang dimaksudkan untuk tahanan geser, atau dengan lembaran baja	7	2½	4½	TB	TB	22	22	22
24. Dinding rangka ringan dengan panel geser dari semua material lainnya	2½	2½	2½	TB	TB	10	TB	TB
25. Rangka baja dengan bresing terkekang terhadap tekuk	8	2½	5	TB	TB	48	48	30
26. Dinding geser pelat baja khusus	7	2	6	TB	TB	48	48	30
C. Sistem rangka pemikul momen								
1. Rangka baja pemikul momen khusus	8	3	5½	TB	TB	TB	TB	TB
2. Rangka batang baja pemikul momen khusus	7	3	5½	TB	TB	48	30	TI
3. Rangka baja pemikul momen menengah	4½	3	4	TB	TB	10^k	TI^k	TI^k
4. Rangka baja pemikul momen biasa	3½	3	3	TB	TB	TI^l	TI^l	TI^l
5. Rangka beton bertulang pemikul momen khusus^m	8	3	5½	TB	TB	TB	TB	TB
6. Rangka beton bertulang pemikul momen menengah	5	3	4½	TB	TB	TI	TI	TI
7. Rangka beton bertulang pemikul momen biasa	3	3	2½	TB	TI	TI	TI	TI
8. Rangka baja dan beton komposit pemikul momen khusus	8	3	5½	TB	TB	TB	TB	TB
9. Rangka baja dan beton komposit pemikul momen menengah	5	3	4½	TB	TB	TI	TI	TI
10. Rangka baja dan beton komposit terkekang parsial pemikul momen	6	3	5½	48	48	30	TI	TI
11. Rangka baja dan beton komposit pemikul momen biasa	3	3	2½	TB	TI	TI	TI	TI
12. Rangka baja canai dingin pemikul momen khusus dengan pembautan^n	3½	3^o	3½	10	10	10	10	10

(Source: SNI 1726:2019 Table 12, p.50)

8. Determination of approach fundamental period, Ta

The coefficient Cu for the upper bound on the calculated period is determined based on the following requirement by SNI 1726:2019.

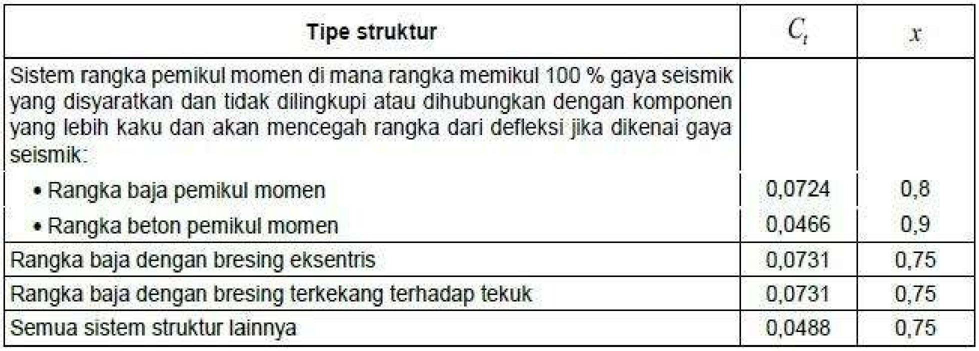
Table 3.10 Cu Coefficient

Parameter percepatan respons spektral desain pada 1 detik, S_{D1}	Koefisien C_u
≥ 0,4	1,4
0,3	1,4
0,2	1,5
0,15	1,6
≤ 0,1	1,7

(Source: SNI 1726:2019 Table 17, p.72)

The parameter values for the approach period, Ct and x are determined based on the following requirement by SNI 1726:2019.

Table 3.11 Parameter Values for the Approach Period, Ct and x



Tipe struktur	C_t	x
Sistem rangka pemikul momen di mana rangka memikul 100 % gaya seismik yang disyaratkan dan tidak dilingkupi atau dihubungkan dengan komponen yang lebih kaku dan akan mencegah rangka dari defleksi jika dikenai gaya seismik:		
• Rangka baja pemikul momen	0,0724	0,8
• Rangka beton pemikul momen	0,0466	0,9
Rangka baja dengan bresing eksentris	0,0731	0,75
Rangka baja dengan bresing terkekang terhadap tekuk	0,0731	0,75
Semua sistem struktur lainnya	0,0488	0,75

(Source: SNI 1726:2019 Table 18, p.72)

The approach period, Ta is calculated using the following equation.

$$Ta = Ct \times hn^{x} \quad (3.15)$$

Meanwhile, the upper bound of the calculated period is determined as Cu×Ta.

9. Calculation of seismic response coefficient, Cs

The seismic response coefficient, Cs can be determined using the following equation.

$$Cs1 = \frac{SDS}{\left(\frac{R}{Ie}\right)} \quad (3.16)$$

For Ta ≤ T_L, it is not necessary for the Cs value to surpass the value obtained from the following equation.

Cs2 $= \frac{\mathrm{SD1}}{\mathrm{Ta}\left(\frac{R}{\mathrm{Ie}}\right)}$ (3.17)

Finally, the Cs value used must not be less than the value obtained from the following equation.

Cs min $= 0.044 \times \mathrm{SDS} \times \mathrm{Ie} \geq 0.01\mathrm{G}$ (3.18)

10. Seismic base shear force (V)

The seismic base shear force (V) in a specified direction shall be determined according to the following equation.

$V = C_x \times W$ (3.19)

11. Lateral seismic force (Fx)

The lateral seismic force (Fx, in kN) at any story must be determined from the following equation.

$F_x = C_{vx} \times V$ (3.20)

$C_{vx} = \frac{W_x {h_x}^k}{\sum_{i=1}^{n} W_i {h_i}^k}$ (3.21)

Meanwhile, the k coefficient value is determined using interpolation. If the structure fundamental period is less than 0.5 s (low-rise building), the k value is 1. If the structure fundamental period is more than 2.5 s (high-rise building), the k value is 2.

12. Horizontal distribution of seismic forces (Vx)

The design seismic story shear at all stories (Vx, in kN), shall be determined from the following equation.

$\mathrm{Vx} = \sum_{i=x}^{n} \mathrm{Fi}$ (3.22)

3.5.2 Response Spectrum

If response spectrum design is needed, the curve design of the response spectrum must be developed by referring to the following figure.

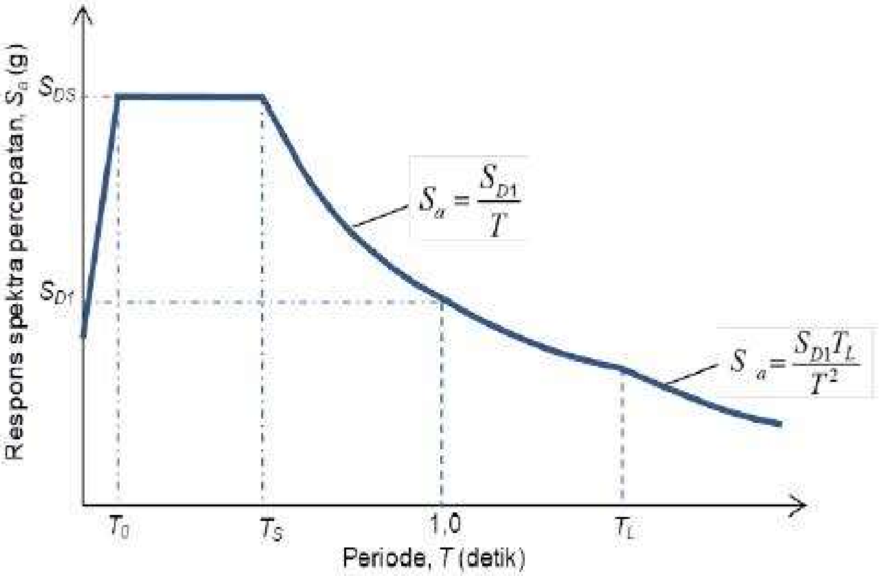


Figure 3.6 Response Spectrum Design

(Source: SNI 1726:2019 Figure 3, p.36)

The requirements for response spectrum acceleration design, Sa are calculated using the following equations.

1. For $T < T_0$

$$Sa = SDS\times\left(0.4 + 0.6\frac{T}{T_0}\right) \quad (3.23)$$

2. For $T \geq T_0$ and $T \leq T_S$

$$Sa = SDS \quad (3.24)$$

3. For $T_S \leq T \leq T_L$

$$Sa = \frac{SD1}{T} \quad (3.25)$$

4. For $T \geq T_L$

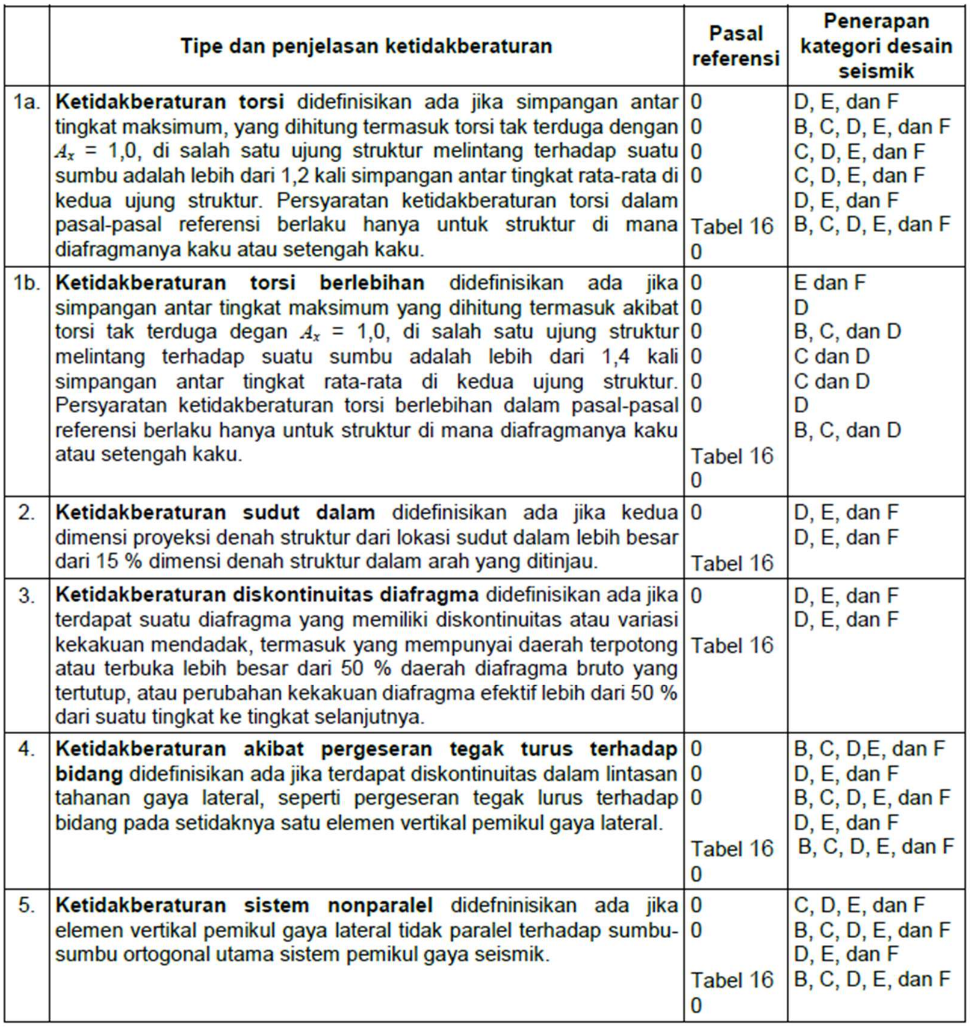
$$Sa = \frac{SD1\times T_L}{T^2} \quad (3.26)$$

3.6 Stage 1 Building Performance under Earthquake Lateral Force (ELF)

3.6.1 Irregularity Analysis

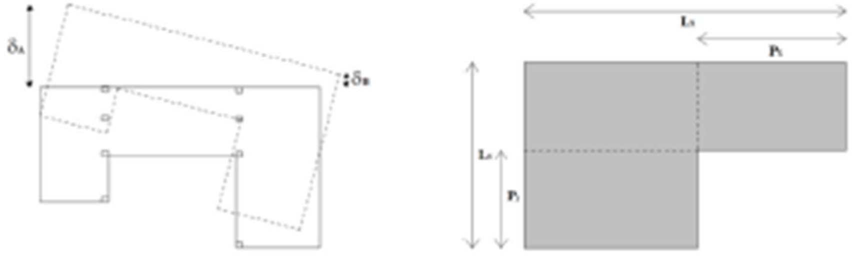
Structures must be classified as either regular or irregular based on the criteria in article 7.3.2 of SNI 1726:2019. The classification shall be based on the horizontal and vertical configuration of the structure. However, in this study, the vertical irregularity analysis is not carried out because the vertical configuration of the building is already considered regular without any height differences. Structures that have one or more types of irregularities as listed in the following table must be declared to have horizontal structural irregularities. Structures designed for the seismic design categories listed in the following table from the SNI (*Standar Nasional Indonesia*) 1726:2019 requirements must comply with the requirements in the articles referred to in the table.

Table 3.12 Structural Horizontal Irregularities



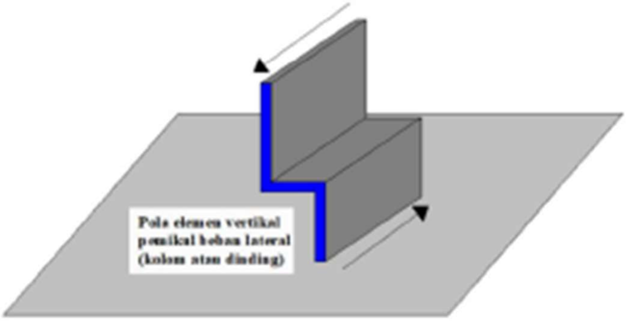
	Tipe dan penjelasan ketidakberaturan	Pasal referensi	Penerapan kategori desain seismik
1a.	**Ketidakberaturan torsi** didefinisikan ada jika simpangan antar tingkat maksimum, yang dihitung termasuk torsi tak terduga dengan A_x = 1,0, di salah satu ujung struktur melintang terhadap suatu sumbu adalah lebih dari 1,2 kali simpangan antar tingkat rata-rata di kedua ujung struktur. Persyaratan ketidakberaturan torsi dalam pasal-pasal referensi berlaku hanya untuk struktur di mana diafragmanya kaku atau setengah kaku.	0 0 0 0 Tabel 16 0	D, E, dan F B, C, D, E, dan F C, D, E, dan F C, D, E, dan F D, E, dan F B, C, D, E, dan F
1b.	**Ketidakberaturan torsi berlebihan** didefinisikan ada jika simpangan antar tingkat maksimum yang dihitung termasuk akibat torsi tak terduga degan A_x = 1,0, di salah satu ujung struktur melintang terhadap suatu sumbu adalah lebih dari 1,4 kali simpangan antar tingkat rata-rata di kedua ujung struktur. Persyaratan ketidakberaturan torsi berlebihan dalam pasal-pasal referensi berlaku hanya untuk struktur di mana diafragmanya kaku atau setengah kaku.	0 0 0 0 0 0 Tabel 16 0	E dan F D B, C, dan D C dan D C dan D D B, C, dan D
2.	**Ketidakberaturan sudut dalam** didefinisikan ada jika kedua dimensi proyeksi denah struktur dari lokasi sudut dalam lebih besar dari 15 % dimensi denah struktur dalam arah yang ditinjau.	0 Tabel 16	D, E, dan F D, E, dan F
3.	**Ketidakberaturan diskontinuitas diafragma** didefinisikan ada jika terdapat suatu diafragma yang memiliki diskontinuitas atau variasi kekakuan mendadak, termasuk yang mempunyai daerah terpotong atau terbuka lebih besar dari 50 % daerah diafragma bruto yang tertutup, atau perubahan kekakuan diafragma efektif lebih dari 50 % dari suatu tingkat ke tingkat selanjutnya.	0 Tabel 16	D, E, dan F D, E, dan F
4.	**Ketidakberaturan akibat pergeseran tegak turus terhadap bidang** didefinisikan ada jika terdapat diskontinuitas dalam lintasan tahanan gaya lateral, seperti pergeseran tegak lurus terhadap bidang pada setidaknya satu elemen vertikal pemikul gaya lateral.	0 0 0 Tabel 16 0	B, C, D,E, dan F D, E, dan F B, C, D, E, dan F D, E, dan F B, C, D, E, dan F
5.	**Ketidakberaturan sistem nonparalel** didefnінisikan ada jika elemen vertikal pemikul gaya lateral tidak paralel terhadap sumbu-sumbu ortogonal utama sistem pemikul gaya seismik.	0 0 Tabel 16 0	C, D, E, dan F B, C, D, E, dan F D, E, dan F B, C, D, E, dan F

(Source: SNI 1726:2019 Table 13, p.59)

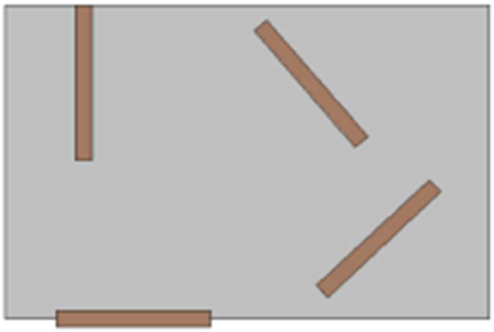


(a) Ketidakberaturan 1a dan 1b (b) Ketidakberaturan 2

(c) Ketidakberaturan 3



(d) Ketidakberaturan 4



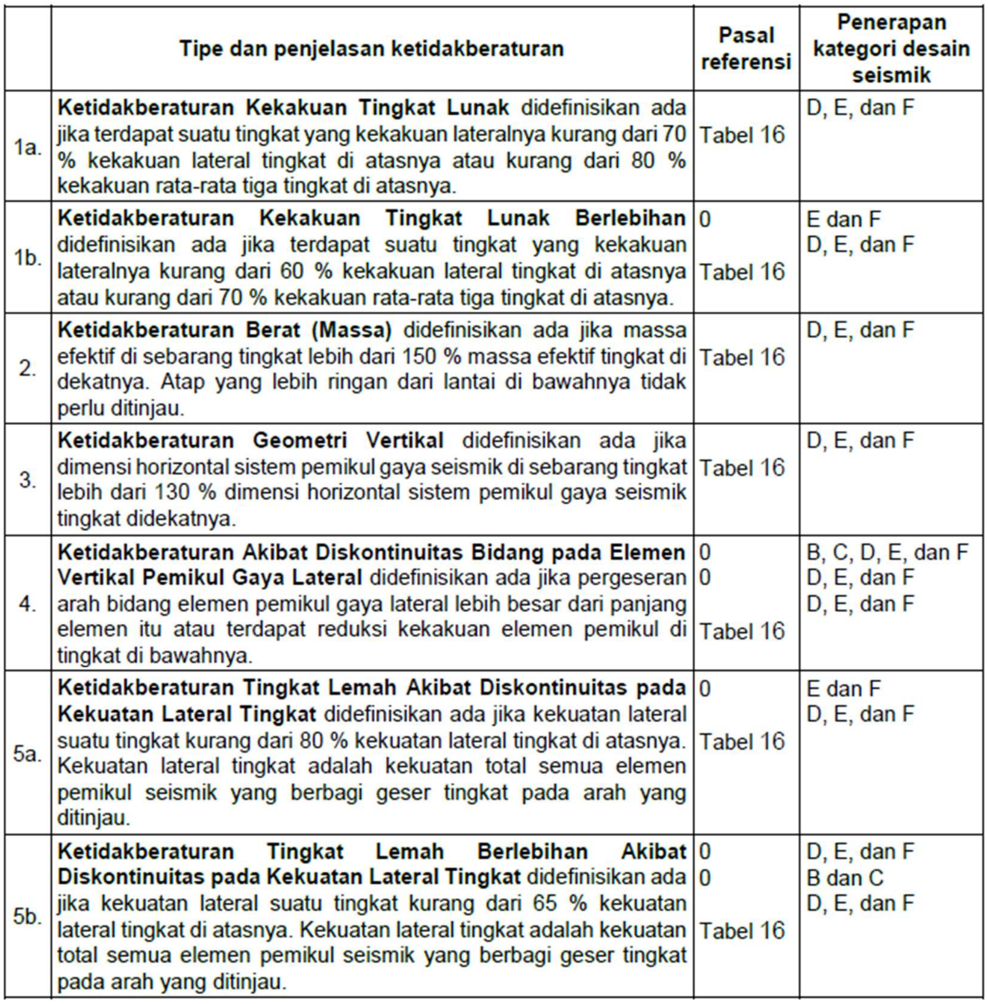
(e) Ketidakberaturan 5

Figure 3.7 Images of Horizontal Irregularities

(Source: SNI 1726:2019 Figure 5, p.60)

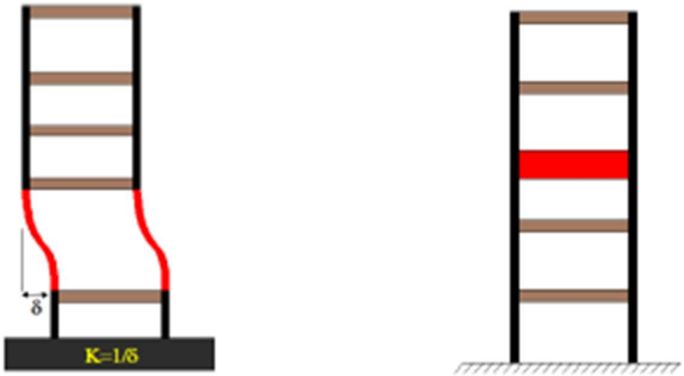
Meanwhile, structures that have one or more types of irregularities as listed in the following table must be declared to have vertical structural irregularities. Structures designed for the seismic design categories listed in the following table from the SNI (*Standar Nasional Indonesia*) 1726:2019 requirements must comply with the requirements in the articles referred to in the table.

Table 3.13 Structural Vertical Irregularities

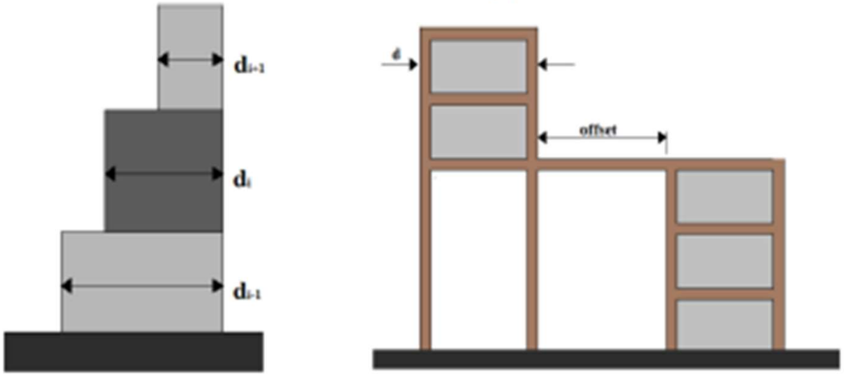


	Tipe dan penjelasan ketidakberaturan	Pasal referensi	Penerapan kategori desain seismik
1a.	**Ketidakberaturan Kekakuan Tingkat Lunak** didefinisikan ada jika terdapat suatu tingkat yang kekakuan lateralnya kurang dari 70 % kekakuan lateral tingkat di atasnya atau kurang dari 80 % kekakuan rata-rata tiga tingkat di atasnya.	Tabel 16	D, E, dan F
1b.	**Ketidakberaturan Kekakuan Tingkat Lunak Berlebihan** didefinisikan ada jika terdapat suatu tingkat yang kekakuan lateralnya kurang dari 60 % kekakuan lateral tingkat di atasnya atau kurang dari 70 % kekakuan rata-rata tiga tingkat di atasnya.	0 Tabel 16	E dan F D, E, dan F
2.	**Ketidakberaturan Berat (Massa)** didefinisikan ada jika massa efektif di sebarang tingkat lebih dari 150 % massa efektif tingkat di dekatnya. Atap yang lebih ringan dari lantai di bawahnya tidak perlu ditinjau.	Tabel 16	D, E, dan F
3.	**Ketidakberaturan Geometri Vertikal** didefinisikan ada jika dimensi horizontal sistem pemikul gaya seismik di sebarang tingkat lebih dari 130 % dimensi horizontal sistem pemikul gaya seismik tingkat didekatnya.	Tabel 16	D, E, dan F
4.	**Ketidakberaturan Akibat Diskontinuitas Bidang pada Elemen Vertikal Pemikul Gaya Lateral** didefinisikan ada jika pergeseran arah bidang elemen pemikul gaya lateral lebih besar dari panjang elemen itu atau terdapat reduksi kekakuan elemen pemikul di tingkat di bawahnya.	0 0 Tabel 16	B, C, D, E, dan F D, E, dan F D, E, dan F
5a.	**Ketidakberaturan Tingkat Lemah Akibat Diskontinuitas pada Kekuatan Lateral Tingkat** didefinisikan ada jika kekuatan lateral suatu tingkat kurang dari 80 % kekuatan lateral tingkat di atasnya. Kekuatan lateral tingkat adalah kekuatan total semua elemen pemikul seismik yang berbagi geser tingkat pada arah yang ditinjau.	0 Tabel 16	E dan F D, E, dan F
5b.	**Ketidakberaturan Tingkat Lemah Berlebihan Akibat Diskontinuitas pada Kekuatan Lateral Tingkat** didefinisikan ada jika kekuatan lateral suatu tingkat kurang dari 65 % kekuatan lateral tingkat di atasnya. Kekuatan lateral tingkat adalah kekuatan total semua elemen pemikul seismik yang berbagi geser tingkat pada arah yang ditinjau.	0 0 Tabel 16	D, E, dan F B dan C D, E, dan F

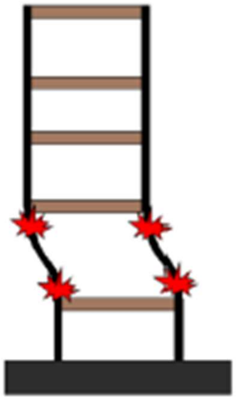
(Source: SNI 1726:2019 Table 14, p.61)



(a) Ketidakberaturan 1a dan 1b (b) Ketidakberaturan 2



(c) Ketidakberaturan 3 (d) Ketidakberaturan 4



(e) Ketidakberaturan 5a dan 5b

Figure 3.8 Images of Vertical Irregularities

(Source: SNI 1726:2019 Figure 6, p.62)

3.6.2 Story Drift

The determination of the story drift (Δ) shall be calculated as the difference in the center of mass deviation above and below the grade under consideration. If the center of mass is not aligned in the vertical direction, it is permissible to calculate the displacement at the bottom of the story based on the vertical projection of the center of mass of the story above. If the allowable stress design is used, Δ shall be calculated using the design seismic force set at 0 without reduction for the allowable stress design.

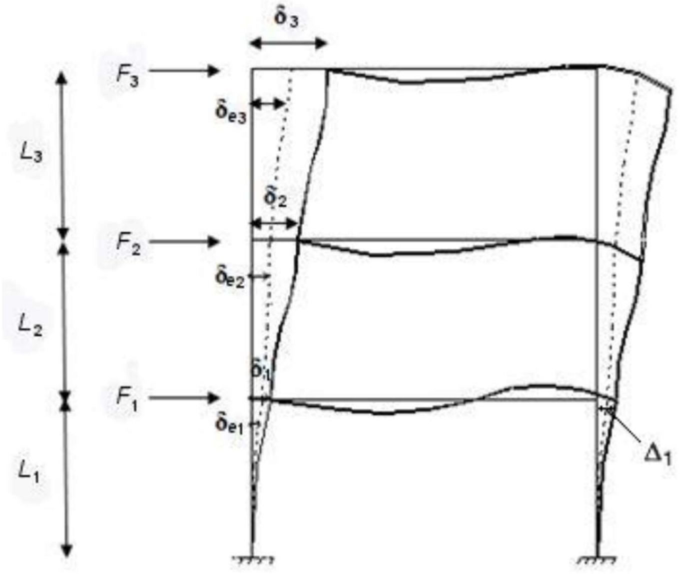


Figure 3.9 Story Drift Determination

(Source: SNI 1726:2019 Figure 10, p.75)

where

δ_i = enlarged displacement = $\frac{Cd \cdot \delta_{ei}}{I_E}$ (3.27)

Δ_i = $\frac{(\delta_{ei} - \delta_{e(i-1)}) \cdot Cd}{I_E} \leq \Delta_a$ (3.28)

Δ_1 = $\delta_{e1} \leq \Delta_a$ (3.29)

with:

F_i = the portion of the seismic shear force at story i

δ_{ei} = elastic displacement calculated due to the design seismic force at story iΔ_i

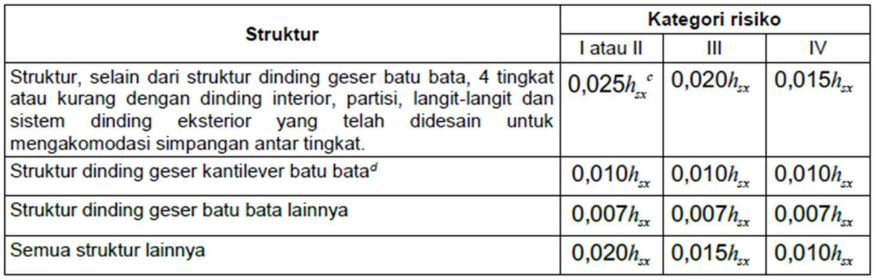
= story drift at story i

Δ_i / L_i = story drift ratio

Δ_a = allowable story drift

Meanwhile, the value of Δ_a or allowable story drift can be determined from the following table.

Table 3.14 Allowable Story Drift



Struktur	Kategori risiko		
	I atau II	III	IV
Struktur, selain dari struktur dinding geser batu bata, 4 tingkat atau kurang dengan dinding interior, partisi, langit-langit dan sistem dinding eksterior yang telah didesain untuk mengakomodasi simpangan antar tingkat.	$0{,}025h_{sx}{}^{c}$	$0{,}020h_{sx}$	$0{,}015h_{sx}$
Struktur dinding geser kantilever batu bata[d]	$0{,}010h_{sx}$	$0{,}010h_{sx}$	$0{,}010h_{sx}$
Struktur dinding geser batu bata lainnya	$0{,}007h_{sx}$	$0{,}007h_{sx}$	$0{,}007h_{sx}$
Semua struktur lainnya	$0{,}020h_{sx}$	$0{,}015h_{sx}$	$0{,}010h_{sx}$

(Source: SNI 1726:2019 Table 20, p.88)

with:

h_{sx} = story height below story x

3.6.3 P-Delta Effect

The effect of P-delta on story shear and moments, the resulting structural member forces and moments, and the resulting story drift need not be considered if the stability coefficient (θ) as determined by the following equation is equal to or less than 0.10:

$$\theta = \frac{P_x \cdot \Delta \cdot I_e}{V_x \cdot h_{sx} \cdot Cd} \tag{3.30}$$

The stability coefficient (θ) must not exceed θ_{max} which is determined as follows:

$$\theta_{max} = \frac{0.5}{\beta \cdot Cd} \leq 0.25 \tag{3.31}$$

with:

P_x = total vertical design load at and above the story x (kN); when calculating, the individual load factor need not exceed 1.0

Δ = deviation between design levels, occurring simultaneously with Vx (mm)

V_x = seismic shear force acting at and above the story x (kN)

β = ratio of the story shear requirement to shear capacity between story x and the story below; this ratio is allowed to be conservatively taken as 1.0

3.7 Stage 2 Computation of Internal Forces for Member Design

3.7.1 Redundancy Factor

A redundancy factor (ρ) shall be assigned to the structure above the isolation system based on the requirements. According to the requirements by SNI 1726:2019, the redundancy factor (ρ) value shall be equal to 1.0 for buildings with a seismic design category of B and C, along with several other requirements presented in Subchapter 7.3.4.1 that must be fulfilled. Meanwhile, the redundancy factor value shall be equal to 1.3 for buildings with a seismic design category of D, E, and F. Despite that, the value may still be considered as 1.0 if the structure fulfils either of the two requirements presented by SNI 1726:2019 in Subchapter 7.3.4.2.

3.7.2 Torsion Analysis

Torsion analysis includes natural torsion and torsion amplification or accidental torsion. The theory based on SNI 1726:2019 is as follows.

1. Natural torsion

   For inflexible diaphragms, the distribution of lateral forces in each story must consider the effect of the inherent natural torsional moment, M_t, due to the eccentricity between the locations of the center of mass and the center of stiffness. For flexible diaphragms, the distribution of forces to the vertical elements must consider the position and distribution of the masses they support.

2. Accidental torsion

   If the diaphragm is inflexible, the design shall include the inherent torsional moment (M_t) resulting from the location of the mass of the structure plus the accidental torsional moment (M_{ta}) due to the displacement of the center of mass from its assumed actual location in each direction within 5% of the vertical structure dimension perpendicular to the direction of the applied force. If seismic forces are applied simultaneously in two orthogonal directions, the required 5% displacement of the center of mass need not be applied in both orthogonal directions at the same time but must be applied in the direction that produces the greater effect.

3.7.3 Loading Scheme

Structures, structural component-elements, and foundation elements shall be designed so that their design strength equals or exceeds the effects of factored loads with the combinations. The effect of having one or more idle loads should be reviewed. The most decisive effects of wind and seismic loads must be considered, but the two loads need not be considered simultaneously.

Based on SNI 1726:2019, the load combinations used for the first stage are as follows.

1. 1.0D + 0.5L + 1.0Ex
2. 1.0D + 0.5L + 1.0Ey

Meanwhile, the load combinations used for the second stage are as follows.

1. 1.2D + 0.5L + 1.0E
2. 0.9D + 1.0E

The load combinations in the second stage consider the substitution of ρ QE ± (0.2SDS) D as a function of E, which can be seen as follows.

1. (1.2 + 0.2SDS) D + 0.5L + ρ QE
2. (0.9 - 0.2SDS) D + ρ QE

The additional 5% eccentricity is also considered in both Ex and Ey in the second stage of structural analysis. The concept of additional eccentricity is explained in the following equations.

$$e_{tx} = 5\% \cdot L_{building} \tag{3.32}$$

$$e_{ty} = 5\% \cdot B_{building} \tag{3.33}$$

$$e_x = e_{ox} \pm e_{tx} \tag{3.34}$$

$$e_y = e_{oy} \pm e_{ty} \tag{3.35}$$

Earthquake loads are a combination of earthquake loads in the X direction and Y direction, either 30% or 100%. The following figure shows an example of these combinations.

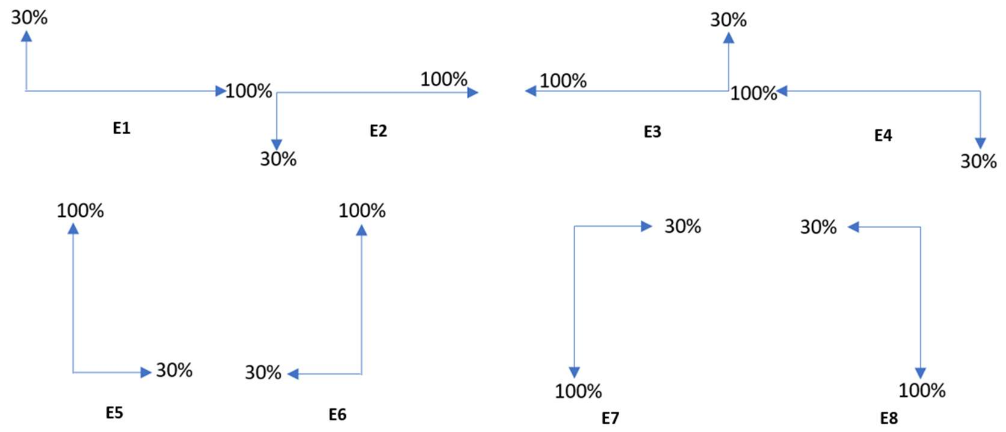


Figure 3.10 Earthquake Load Combinations of Ex and Ey

The final load combinations for the second stage are as follows.

1. (1.2 + 0.2SDS) D + 0.5L + ρ QEx+TT + ρ 30%QEy
2. (1.2 + 0.2SDS) D + 0.5L + ρ QEx+TT - ρ 30%QEy
3. (1.2 + 0.2SDS) D + 0.5L - ρ QEx+TT + ρ 30%QEy
4. (1.2 + 0.2SDS) D + 0.5L - ρ QEx+TT - ρ 30%QEy
5. (1.2 + 0.2SDS) D + 0.5L + ρ QEx-TT + ρ 30%QEy
6. (1.2 + 0.2SDS) D + 0.5L + ρ QEx-TT - ρ 30%QEy
7. (1.2 + 0.2SDS) D + 0.5L - ρ QEx-TT + ρ 30%QEy
8. (1.2 + 0.2SDS) D + 0.5L - ρ QEx-TT - ρ 30%QEy
9. (1.2 + 0.2SDS) D + 0.5L + ρ QEy+TT + ρ 30%QEx
10. (1.2 + 0.2SDS) D + 0.5L + ρ QEy+TT - ρ 30%QEx
11. (1.2 + 0.2SDS) D + 0.5L - ρ QEy+TT + ρ 30%QEx
12. (1.2 + 0.2SDS) D + 0.5L - ρ QEy+TT - ρ 30%QEx
13. (1.2 + 0.2SDS) D + 0.5L + ρ QEy-TT + ρ 30%QEx
14. (1.2 + 0.2SDS) D + 0.5L + ρ QEy-TT - ρ 30%QEx
15. (1.2 + 0.2SDS) D + 0.5L - ρ QEy-TT + ρ 30%QEx
16. (1.2 + 0.2SDS) D + 0.5L - ρ QEy-TT - ρ 30%QEx
17. (0.9 - 0.2SDS) D + ρ QEx+TT + ρ 30%QEy
18. (0.9 - 0.2SDS) D + ρ QEx+TT - ρ 30%QEy
19. (0.9 - 0.2SDS) D - ρ QEx+TT + ρ 30%QEy
20. (0.9 - 0.2SDS) D - ρ QEx+TT - ρ 30%QEy

21. (0.9 - 0.2SDS) D + ρ QEx-TT + ρ 30%QEy
22. (0.9 - 0.2SDS) D + ρ QEx-TT - ρ 30%QEy
23. (0.9 - 0.2SDS) D - ρ QEx-TT + ρ 30%QEy
24. (0.9 - 0.2SDS) D - ρ QEx-TT - ρ 30%QEy
25. (0.9 - 0.2SDS) D + ρ QEy+TT + ρ 30%QEx
26. (0.9 - 0.2SDS) D + ρ QEy+TT - ρ 30%QEx
27. (0.9 - 0.2SDS) D - ρ QEy+TT + ρ 30%QEx
28. (0.9 - 0.2SDS) D - ρ QEy+TT - ρ 30%QEx
29. (0.9 - 0.2SDS) D + ρ QEy-TT + ρ 30%QEx
30. (0.9 - 0.2SDS) D + ρ QEy-TT - ρ 30%QEx
31. (0.9 - 0.2SDS) D - ρ QEy-TT + ρ 30%QEx
32. (0.9 - 0.2SDS) D - ρ QEy-TT - ρ 30%QEx

3.7.4 Equivalent Lateral Force (ELF) Analysis

Equivalent static analysis is a structural analysis method with earthquake vibrations which are modelled as static horizontal loads acting on the building's mass centers. In buildings with many masses, there will be many horizontal forces each acting on these masses. Following the principle of balance, it can be analogous to the existence of a horizontal force acting on the base of the building, which is then called the base shear force, V. This basic shear force forms a balance with the horizontal force acting on each mass of the building (Pawirodikromo, Respons Dinamik Struktur Elastik, 2001).

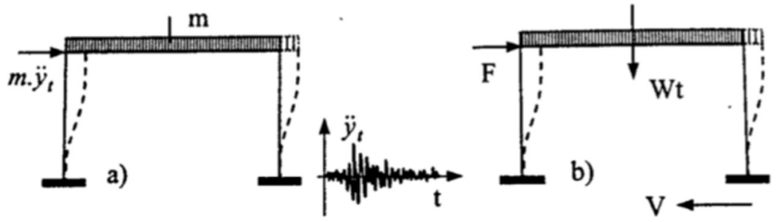


Figure 3.11 a) Dynamic Analysis, b) Horizontal Static Equivalent Force

(Source: Pawirodikromo, 2012)

In the dynamic analysis as shown in Figure a), the vibration/sway of the building is caused by the ground vibration load in the form of an accelerogram. For reasons previously mentioned, the dynamic load effect is then simplified into a load acting on the center of mass. The horizontal force acting on the building's mass centers is only static in nature, meaning that the magnitude and location are fixed, while the dynamic load varies in intensity according to time (dynamic). These horizontal forces are only equivalent in characteristic as a substitute/representation of the dynamic load effect that occurs during an earthquake. Therefore, these horizontal forces are generally referred to as static equivalent horizontal forces/loads.

The seismic base shear force (V) in a specified direction shall be determined according to Equation 3.19 as previously mentioned.

3.7.5 Base Shear Scaling

If the fundamental period of the analysis results is greater than CuTa in a certain direction, then the period of the T structure must be taken as equal to CuTa. If the combined response for the base shear force resulting from the analysis of variance (V_t) is less than 100% of the shear force (V) calculated using the equivalent static method, then the force must be multiplied by V/V_t, where V is the calculated equivalent static base shear according to SNI 1726:2019, and V_t is the base shear force obtained from the analysis of the combination of variances.

In the ETABS model itself, the scale factor input is determined by the value of I/R including the multiplication by gravitational acceleration (9806.65 mm/s^2). This value is then inserted into the load case scale factor of the response spectrum in both X and Y directions.

3.7.6 Response Spectrum (RS) Analysis

Dynamic structural analysis includes time history analysis (THA) and response spectrum (RS) analysis, which procedure will be used in this study. Before carrying out a dynamic analysis of the response spectrum, modal analysis must be carried out first. Modal analysis is carried out to determine the elastic period and the range of vibrations produced by a structure or building when subjected to an earthquake force. The modal analysis consists of two types, namely Eigenvectors and Ritz vectors. In this study, the modal analysis used is the Eigenvector type.

Eigenvector analysis produces the free vibrational mode without damping and frequency of the system. From the variety of vibrations, the behavior of a structure when experiencing an earthquake force can be seen (Wantalangie, Pangouw, & Windah, 2016).

In the variance response parameter, the value for each design parameter related to the force under consideration, including story drift, support force, and individual structural member forces for each response mode must be calculated using the properties of each mode and the response spectrum defined in SNI 1726:2019 divided by the quantity (R/Ie). The values for the displacements and the quantity of drift between levels must be multiplied by the quantity (Cd/Ie).

For the combined response parameters, the values for each parameter under review, which are calculated for various variances, must be combined using the square root sum of squares method (SRSS) or the complete squares combination method (CQC), according to SNI 1726:2019. The CQC method should be used for each of the variance values where the adjacent variance has a significant cross-correlation between the translational and torsional responses.

3.8 Structural Member Reinforcement Design

Earthquake-resistant building is defined as a building that meets the criteria of being rather economical according to certain standards and is resistant to earthquake loads with a certain fundamental period, earthquake intensity, and ground acceleration, which will keep the building occupants safe according to those criteria. As the building in this study is earthquake-resistant, the structural members must also be designed to withstand gravity and earthquake loads. The design of the structural members is elaborated in the following subchapters.

3.8.1 Main Beam Reinforcement Design

The main beam reinforcements are designed according to the area of support and middle span of the beams. The support area starts from the edge of the beam until a quarter (1/4) of the beam length on both sides, while the middle part of the beam is called the middle span area. The ultimate moments withstanding the gravity and earthquake loads must be checked whether the percentage between the negative and positive moments has surpassed the ratio of 50%. If the ultimate moments have not surpassed 50%, the moments must be redistributed. The redistributed ultimate moments can be used to design the reinforcement of the main beams.

In the support area of the beam, the moment used is the negative moment, while the positive moment is used for the middle span area of the beam. Once the moments are redistributed, the flexural and shear (stirrup) reinforcements can be designed. However, it is noted that the beam reinforcement design need not be carried out differently for each span and at each story. Moments that are rather close in value can be taken to be the same, so that there may be several groups of moments for the entire height of the building. The other structural member designs are also carried out in the same way.

The beams of a Special Moment Resisting Frame Structure (SRPMK) are designed according to SNI 2847:2019 Article 18.6. According to Article 18.6.2, the dimensions of the beam must fulfill the following requirements.

1. $\mathrm{Ln} \geq 4\mathrm{D}_{\mathrm{flexural}}$
2. $\mathrm{B} \geq 0.3\mathrm{H}$ or $\mathrm{B} \geq 250$ mm (the smaller value)

If the dimension has fulfilled the requirements, the next process is to design flexural reinforcements for the main beams. The assumption of the number of reinforcements needed is analyzed as follows.

The number of reinforcements used and the spacing are determined using the following equations.

$$\mathrm{Mn}_1 = \varphi \cdot \mathrm{Mu} \cdot \mathrm{R} \tag{3.36}$$

$$\mathrm{Cc} = 0.85 \cdot \mathrm{f'c} \cdot \mathrm{a} \cdot \mathrm{b} \tag{3.37}$$

$$\mathrm{Mn} = \mathrm{Cc}\left(\mathrm{d} - \frac{\mathrm{a}}{2}\right) \tag{3.38}$$

$$\mathrm{As}_1 = \frac{\mathrm{Cc}}{\mathrm{fy}} \tag{3.39}$$

$$\mathrm{Mn}_2 = \mathrm{Mu} - \mathrm{Mn}_1 \tag{3.40}$$

$$\mathrm{Ts} = \frac{\mathrm{Mn}_2}{\mathrm{fy}} \tag{3.41}$$

$$\mathrm{As}_2 = \frac{\mathrm{Ts}}{\mathrm{fy}} \tag{3.42}$$

$$\mathrm{n} = \frac{\mathrm{As}}{\mathrm{As}_{1\mathrm{D}}} \tag{3.43}$$

$$s = \frac{B-(2\cdot\text{Concrete cover})-(2\cdot Ds)-(n\cdot Dp)}{n-1} \quad (3.44)$$

with:

Mn = nominal moment

Mu = ultimate moment

φ = strength reduction factor

R = flexural resistance factor

Cc = compressive force of concrete

Ts = tensile force of reinforcement

a = concrete compression thickness

As = cross-section area

f’c = compressive strength of concrete

fy = yield strength of steel

s = spacing between reinforcements

Ds = diameter of shear reinforcement

Dp = diameter of flexural reinforcement

The nominal and probable moment analysis of the main beams is determined according to the following equations. The main difference in probable moment analysis is that the parameter of the structural overstrength factor ($ø_{os}$) is considered.

$$As = n \cdot As_{1D} \quad (3.45)$$

$$As \cdot fy = 0.85 \cdot f'c \cdot \beta \cdot b + \left(\frac{c-ds'}{c}\right) \cdot \varepsilon c \cdot Es \cdot As' \quad (3.46)$$

$$a = c \cdot \beta \quad (3.47)$$

$$fs = \left(\frac{c-ds'}{c}\right) \cdot \varepsilon c \cdot Es \quad (3.48)$$

$$Mn_{cc} = 0.85 \cdot f'c \cdot a \cdot b \cdot \left(d - \frac{a}{2}\right) \quad (3.49)$$

$$Mn_{cs} = As' \cdot fs \cdot (d - ds') \quad (3.50)$$

$$Mn = Mn_{cc} + Mn_{cs} \quad (3.51)$$

with:

β = concrete strength parameter

fs = compressive steel stress

εc = compressive strain

Es = elastic modulus of steel

$\mathrm{Mn}_{\mathrm{cc}}$ = nominal moment of compressive force of concrete

$\mathrm{Mn}_{\mathrm{cs}}$ = nominal moment of compressive force of reinforcement

The analysis of flexural reinforcement is carried out in both support and middle span areas. The difference of the number of reinforcement (n) between support and middle span area is as follows.

1. Support area: Upper reinforcement area is the negative or tensile area, and lower reinforcement area is the positive or compression area.
2. Middle span area: Upper reinforcement area is the positive or compression area, and lower reinforcement area is the negative or tensile area.

Apart from bending moment, shear force is also one of the dominant forces that occurs in beams. It is important to design the beams to be able to withstand the shear forces acting upon them. Meanwhile, the axial force in the beam is usually relatively small and can be ignored. Torsion moments may also occur under certain conditions.

For the shear reinforcement design of main beams, the shear force values obtained from the analysis of the ETABS model are the shear force due to gravitational load (Vg) within and outside the plastic joint area (Lo). Meanwhile, the shear force due to earthquake load (Ve) may be determined using the previously determined values of beam probable moments (Mpr). The ultimate shear force (Vu) may then be determined by adding the values of Vg and Ve. The equations used in this analysis are as follows.

$$\mathrm{Vg}_{\mathrm{used}} = \frac{\mathrm{Vg}_{\mathrm{ETABS}}}{\varphi} \tag{3.52}$$

$$\mathrm{Ve} = \left(\frac{\mathrm{Mpr}^{-}}{\mathrm{L}_{\mathrm{netto}} \cdot \varphi} + \frac{\mathrm{Mpr}^{+}}{\mathrm{L}_{\mathrm{netto}} \cdot \varphi}\right) \tag{3.53}$$

$$\mathrm{Vu} = \mathrm{Vg} + \mathrm{Ve} \tag{3.54}$$

with:

Vg = shear force due to gravitational load

Ve = shear force due to earthquake load

Vu = ultimate shear force

L_{netto} = net length of beam

As the Shear Force Diagram (SFD) and Free Body Diagram (FBD) are obtained from the data, the dimensions of the diagrams are analyzed to determine the nominal shear strength provided by shear reinforcement (Vs) as well as the nominal shear strength provided by concrete (Vc). The dimensions of the shear force diagrams are determined as follows.

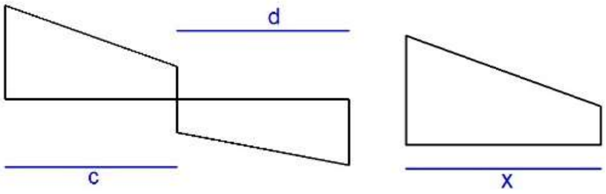


Figure 3.12 Illustration of the SFD Dimension

$$c = \frac{Vu_{left} - Vu_{upper}}{(Vu_{left} - V\ _{upper}) + (Vu_{right} - Vu_{lower})} \cdot L_{netto} \quad (3.55)$$

$$d = L_{netto} - c \quad (3.56)$$

The equation used to determine the nominal shear strength provided by concrete (Vc) is as follows.

$$Vc = \frac{1}{6} \cdot \sqrt{f'c} \cdot B \cdot H^{-} \quad (3.57)$$

If the value of Ve > Vg_{right}, the Vs_1 value, which is the nominal shear strength provided by shear reinforcement in the plastic joint area (Lo) is determined as the bigger value between Vu_{left} and Vu_{right}. Meanwhile, if it is the other way around, the Vs_1 value is subtracted by the value of Vc. Outside of the plastic joint area (outside of Lo), the Vs_2 value is determined by subtracting the Vc value by the y value, which equation is as follows.

$$V_{ats} = Vu_{max\,left/right} - Ve - Vg_{max\,upper/lower} \quad (3.58)$$

$$x = \frac{V_{ats} \cdot (x - H_{beam})}{x} \quad (3.59)$$

$$y = x + Vg_{upper} + Ve \quad (3.60)$$

The x value is determined from the following figure.

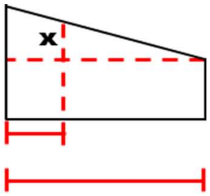


Figure 3.13 Illustration of the SFD for the Outside of Plastic Joint Area Analysis

After determining the number (n) of shear reinforcements (stirrups) needed, the spacing (s) is determined using the following equation.

$$s = n_{stirrup} \cdot \frac{Av \cdot fy_{stirrup} \cdot H^{-}}{Vs_{1}} \qquad (3.61)$$

with:

Vs_1 = nominal shear strength provided by shear reinforcement in the plastic joint area (Lo)

Vs_2 = nominal shear strength provided by shear reinforcement outside of the plastic joint area (outside of Lo)

Av = cross-section area of shear reinforcement

Furthermore, the spacing of the plastic joint area (Lo) is checked according to SNI 2847:2019 to not surpass the minimum value of the following requirements.

1. $\frac{H^{-}}{4}$
2. $8 \cdot D_{flexural}$
3. $24 \cdot D_{shear}$
4. $300\ \text{mm}$

Meanwhile, the spacing outside of the plastic joint area (outside of Lo) is checked according to not surpass the value of $\frac{H^{-}}{2}$ and to have a minimum spacing of 50 mm.

3.8.2 Secondary Beam Reinforcement Design

The reinforcement design of the secondary beam is theoretically similar to the main beam, hence the equations used have been mentioned in the previous subchapter.

3.8.3 Column Reinforcement Design

The reinforcement design of the column element includes flexural and shear reinforcement design with the checking of Strong Column Weak Beam (SCWB) requirement, as well as the beam-column joint (BCJ) reinforcement design. The equations used in designing the flexural reinforcement are as follows.

$$\mathrm{Ag} = \mathrm{Ht} \times \mathrm{B} \tag{3.62}$$

$$\mathrm{As}_{\mathrm{flexural}} = \frac{1}{4} \cdot \pi \cdot \mathrm{D}_{\mathrm{flexural}}{}^{2} \tag{3.63}$$

$$\mathrm{As}_{\mathrm{shear}} = \frac{1}{4} \cdot \pi \cdot \mathrm{D}_{\mathrm{shear}}{}^{2} \tag{3.64}$$

$$\varepsilon_{\mathrm{y}} = \frac{\mathrm{fy}}{\mathrm{Es}} \tag{3.65}$$

$$\mathrm{As}_{\mathrm{needed}} = \mathrm{Ag\ and\ reinforcemet\ ratio} \times \mathrm{Ag} \tag{3.66}$$

$$\mathrm{n}_{\mathrm{needed}} = \frac{\mathrm{As}_{\mathrm{needed}}}{\mathrm{As}_{\mathrm{flexural}}} \tag{3.67}$$

with:

Ag = cross-section area of column

$\mathrm{As}_{\mathrm{flexural}}$ = cross-section area of flexural reinforcement

$\mathrm{As}_{\mathrm{shear}}$ = cross-section area of shear reinforcement

ε_{y} = steel tensile strain

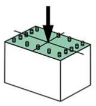
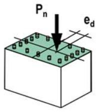
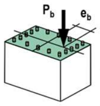
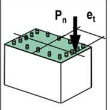
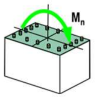
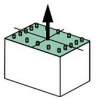
Ratio = 1.5%

$\mathrm{As}_{\mathrm{needed}}$ = total cross-section area of flexural reinforcements needed

$\mathrm{n}_{\mathrm{needed}}$ = number of flexural reinforcements needed

From the number of reinforcements (n) needed that was obtained, the Mn-Pn diagram for both X and Y portals is determined to obtain the nominal moments (Mn), which will then be used for the SCWB checking. The considered column conditions for the Mn-Pn diagram analysis are as follows.

Table 3.15 The Considered Column Conditions for Mn-Pn Diagram Analysis

1		**Centric load**, i.e. P load is acting exactly in the center of the column section.
2		**Compression failure** is a condition where the nominal column axial load Pn is greater than Pb, because the load acting with smaller eccentricity than balance eccentricity (e < eb).
3		**Balance** condition is a condition where by a certain Pnb, eccentricity (eb) and reinforcements (Asb), the steel is yielding at the same time as concrete compression strain reaching the maximum value.
4		**Tension failure** is a condition where Pn is relatively small (Pn < Pb) and is acting on the column with a relatively large eccentricity (e > eb).
5		**Pure bending** condition is the condition where the axial force of the column is zero. The column acting similar as a beam.
6		**Pure tensile** condition is a condition where the column is acting fully as tension column.

According to the conditions mentioned above, the equations used to analyze the Mn-Pn diagram are as follows.

For centric load condition, the equations are as follows.

$$\mathrm{Cc} = 0.85 \cdot \mathrm{f'c} \cdot \mathrm{B} \cdot \mathrm{Ht} \tag{3.68}$$

$$\mathrm{Cs}_1 = \mathrm{As}(\mathrm{fy} - 0.85 \cdot \mathrm{f'c}) \tag{3.69}$$

$$\mathrm{Cs}_2 = \mathrm{As'}(\mathrm{fy} - 0.85 \cdot \mathrm{f'c}) \tag{3.70}$$

$$\mathrm{Pn} = \mathrm{Cc} + \mathrm{Cs}_1 + \mathrm{Cs}_2 \tag{3.71}$$

In the centric load condition, the value of the nominal moment (Mn) is considered as 0 ton-m.

For compression and tensile failure conditions, the equations are as follows. The difference between these conditions is the number of n used to determine the value of C. The compression failure condition uses the values above 1, whereas the tensile failure condition uses the values under 1.

$$\mathrm{C} = \mathrm{n} \cdot \mathrm{Cb} \tag{3.72}$$

$$\mathrm{a} = \beta \cdot \mathrm{C} \tag{3.73}$$

$$\varepsilon_s' = \frac{C - d\prime}{C} \cdot 0.003 \tag{3.74}$$

To check the compression steel strain, the value of $\varepsilon_s' > \varepsilon_y$ means that the steel has yielded, otherwise it has not. Meanwhile, to check the tension steel strain, the value of $\varepsilon_s > \varepsilon_y$ means that the steel has yielded, otherwise it has not. The ε_s value is calculated with the following equation.

$$\varepsilon_s = \frac{h - C}{C} \cdot 0.003 \tag{3.75}$$

$$Cc = 0.85 \cdot f'c \cdot a \cdot B \tag{3.76}$$

$$Cs = As'(fy - 0.85 \cdot f'c) \tag{3.77}$$

$$Ts = As \cdot fs \tag{3.78}$$

$$Pn = Cc + Cs - Ts \tag{3.79}$$

$$Mn = Cc\left(\frac{1}{2}Ht - \frac{1}{2}a\right) + Cs\left(\frac{1}{2}Ht - d'\right) + Ts\left(\frac{1}{2}Ht - d\right) d \tag{3.80}$$

For balance condition, the equations are as follows. The value of Cb is also used to determine the C values of both compression and tensile failure conditions.

$$Cb = \frac{\varepsilon_c}{\varepsilon_c + \varepsilon_y} \cdot H \tag{3.81}$$

$$ab = \beta \cdot Cb \tag{3.82}$$

$$Cc = 0.85 \cdot f'c \cdot ab \cdot B \tag{3.83}$$

$$Cs = As'(fy - 0.85 \cdot f'c) \tag{3.84}$$

$$Ts = As \cdot fy \tag{3.85}$$

$$Pnb = Cc + Cs - Ts \tag{3.86}$$

$$Mnb = Cc\left(\frac{1}{2}Ht - \frac{1}{2}ab\right) + Cs\left(\frac{1}{2}Ht - d'\right) + Ts\left(\frac{1}{2}Ht - d\right) \tag{3.87}$$

For pure bending condition, the equation of quadratic formula is used. The equations used to analyze the quadratic formula are as follows.

$$xa^2 + ya - z = 0 \tag{3.88}$$

$$x = 0.85 \cdot f'c \cdot B \tag{3.89}$$

$$y = As' \cdot \varepsilon_{cu} \cdot Es - As \cdot fy \tag{3.90}$$

$$z = As' \cdot \varepsilon_{cu} \cdot Es \cdot \beta \cdot d' \tag{3.91}$$

$$a = \frac{-y \pm \sqrt{y^2 - 4xz}}{2x} \tag{3.92}$$

$$C = \frac{a}{\beta} \tag{3.93}$$

$$\varepsilon_s = \frac{C-d}{C} \cdot \varepsilon_c \tag{3.94}$$

$$fs = \varepsilon_s \cdot E \tag{3.95}$$

$$Mn = Cc\left(h - \frac{a}{2}\right) + Ts(h - d') \tag{3.96}$$

In the pure bending condition, the value of the nominal axial load (Pn) is considered as 0 ton.

For pure tensile condition, the equation for the tensile axial load is as follows.

$$Pt = -(As + As')fy \tag{3.97}$$

In the pure tensile condition, the value of the tensile moment (Mt) is considered as 0 ton-m.

With the axial loads and moments from each condition are obtained, the Mn-Pn diagram can be established. With the ultimate moment (Mu) values obtained from the ETABS model analysis, the Mn and Pn values can be determined.

To confirm whether the nominal moment of the columns have completed the requirements of Strong Column Weak Beam (SCWB), the nominal moment of the columns must be larger than 1.2 times of the total nominal moment of both the left and right beams intersecting the column in each portal. Therefore, the nominal moment of beams previously obtained is incorporated into the SCWB analysis. If the nominal moment of columns (in upper and lower story) in every story have surpassed 1.2 times of the total nominal moment of beams (located in left and right of the column), the SCWB requirement is confirmed, and the number of flexural reinforcements previously designed may be used. Otherwise, the number must be increased.

Meanwhile, the shear reinforcement design of the columns differs in the first story from the other stories because plastic joint happens in the first story. Therefore, the shear reinforcement design of the first story also differs from the other stories. Before designing the shear reinforcement of columns, the shear forces due to gravity and earthquake loads are determined beforehand. The equations are as follows.

$$Vu = \frac{Muc_{upper} + Muc_{lower}}{L_{column}} \tag{3.98}$$

$$Mprc = \alpha(Mpr^{-}b + Mpr^{+}b) \tag{3.99}$$

$$\mathrm{Ve} = \frac{\mathrm{Mprc}_{\mathrm{upper}} + \mathrm{Mprc}_{\mathrm{lower}}}{\mathrm{L}_{\mathrm{column}}} \tag{3.100}$$

with:

Muc = ultimate moment of column

Mprc = probable moment of column

Mprb = probable moment of beam

α = coefficient of 0.5 for regular stories, 1.0 for the top story

The equations used for shear reinforcement design of the columns in the first story are as follows.

$$\mathrm{h}_{\mathrm{X\ direction}} = \mathrm{B} - \mathrm{Sb} - \mathrm{D}_{\mathrm{shear}} - \left(\frac{\mathrm{D}_{\mathrm{flexural}}}{2}\right) \tag{3.101}$$

$$\mathrm{h}_{\mathrm{Y\ direction}} = \mathrm{Ht} - \mathrm{Sb} - \mathrm{D}_{\mathrm{shear}} - \left(\frac{\mathrm{D}_{\mathrm{flexural}}}{2}\right) \tag{3.102}$$

$$\mathrm{Ln}_{\mathrm{column}} = \mathrm{L}_{\mathrm{column}} - \frac{\mathrm{H}_{\mathrm{upper\ beam}}}{2} - \frac{\mathrm{H}_{\mathrm{lower\ beam}}}{2} \tag{3.103}$$

$$\mathrm{Hx} = \mathrm{B} - 2 \cdot \mathrm{Sb} \tag{3.104}$$

$$\mathrm{Hy} = \mathrm{Ht} - 2 \cdot \mathrm{Sb} \tag{3.105}$$

$$\mathrm{Ach} = \mathrm{Hx} \times \mathrm{Hy} \tag{3.106}$$

with:

Sb = concrete cover thickness

Ln = net length

Ach = cross-section area of the structural component measured to the outer edge of the transverse (shear) reinforcement

The length of the plastic joint (Lo) must not surpass the following values as a requirement according to SNI 2847:2019.

1. $\frac{1}{6} \cdot \mathrm{Ln}$
2. Maximum column dimension
3. 450 mm

The spacing checking according to SNI 2847:2019 is as follows.

1. $\frac{1}{4} \cdot$ Minimum column dimension
2. $\frac{1}{6} \cdot$ Minimum $\mathrm{D}_{\mathrm{flexural}}$
3. 100 mm

The total cross-section area (Ash) of the transverse (shear) reinforcement is analyzed in both X and Y directions to determine the number of shear reinforcements (n) needed. The equations for the plastic joint area (Lo) according to SNI 2847:2019 are as follows.

$$\mathrm{Ash_1} = 0.3\left(\mathrm{s} \cdot \mathrm{Bc} \cdot \frac{\mathrm{f'c}}{\mathrm{fyt}}\right)\left(\frac{\mathrm{Ag}}{\mathrm{Ach}} - 1\right) \tag{3.107}$$

$$\mathrm{n_1} = \frac{\mathrm{Ash_1}}{\mathrm{Ad}} \tag{3.108}$$

$$\mathrm{Ash_2} = 0.9\left(\mathrm{s} \cdot \mathrm{Bc} \cdot \frac{\mathrm{f'c}}{\mathrm{fyt}}\right) \tag{3.109}$$

$$\mathrm{n_2} = \frac{\mathrm{Ash_2}}{\mathrm{Ad}} \tag{3.110}$$

The equation used to determine the spacing between the legs of the stirrups or shear reinforcements in both the X and Y directions is as follows.

$$\mathrm{x_1} = \frac{\left(\mathrm{B} - 2 \cdot \mathrm{Sb} - 2 \cdot \frac{\mathrm{D_{shear}}}{2}\right)}{\mathrm{n} - 1} \tag{3.111}$$

Furthermore, the spacing of the shear reinforcement design outside of the plastic joint area (outside of Lo) according to SNI 2847:2019 is checked so that it does not surpass h/2.

Meanwhile, the equations used for shear reinforcement design of the columns in other stories are as follows.

$$\mathrm{Vc} = 0.17\left(1 + \frac{\mathrm{Nu}}{14 \cdot \mathrm{Ag}}\right)\lambda \cdot \sqrt{\mathrm{f'c}} \cdot \mathrm{B} \cdot h \tag{3.112}$$

$$\mathrm{Vs} = \mathrm{Vn} - \mathrm{Vc} \tag{3.113}$$

$$\mathrm{Av} = \mathrm{n} \cdot \mathrm{Ad} \tag{3.114}$$

$$\mathrm{s_{max}} = \frac{\mathrm{Av} \cdot \mathrm{fy} \cdot \mathrm{h}}{\mathrm{Vs}} \tag{3.115}$$

with:

Av = total cross-section area of shear reinforcements

$\mathrm{s_{max}}$ = maximum tolerable spacing between reinforcements

The length of the plastic joint (Lo) must not surpass the following values as a requirement according to SNI 2847:2019.

1. $8 \cdot \mathrm{Minimum\ D_{flexural}}$
2. $24 \cdot \mathrm{D_{shear}}$

3. $\frac{1}{2} \cdot \text{Minimum column dimension}$
4. $300\ \text{mm}$

The next analysis is carried out for the beam-column joint (BCJ) of each story. The beam-column joint plays an important role in the stability of the structure. Priestley & Paulay (1992) stated that the main problems in the beam-column joints are as follows.

1. Horizontal and vertical shear forces may be several times greater than the shear force at adjacent beams and columns.
2. Joint stress problems arise due to the combination of compression and tension within the reinforcement line.

As beam-column joint is considered a plastic joint, the design of BCJ is similar to the shear reinforcement design in the plastic joint area (Lo) in the first story. The beam-column joint shear stress in all BCJ of each story in both X and Y directions must then be checked if the reinforcements and the column dimensions have fulfilled the requirements of SNI 2847:2019. The equations used are as follows.

$$\mathrm{Aj} = \left(\frac{\mathrm{B_{column}} - \mathrm{B_{beam}}}{2} + \mathrm{B_{beam}}\right) \mathrm{H_{column}} \tag{3.116}$$

$$\mathrm{Vn} = 1.7 \cdot \sqrt{\mathrm{f'c}} \cdot \mathrm{Aj} \tag{3.117}$$

$$\mathrm{V_{column}} = \frac{\varphi\left(\frac{\mathrm{Lb_{left}}}{\mathrm{Lbn_{left}}}\right)\mathrm{Mpr^{-}}_{\mathrm{left}} + \left(\frac{\mathrm{Lb_{right}}}{\mathrm{Lbn_{right}}}\right)\mathrm{Mpr^{+}}_{\mathrm{right}}}{\frac{1}{2}\left(\mathrm{h_{upper\ column}} + \mathrm{h_{lower\ column}}\right)} \tag{3.118}$$

$$\mathrm{Ts} = \frac{\varphi \cdot \mathrm{Mpr^{-}}_{\mathrm{left}}}{\mathrm{h_{left}} - \frac{\mathrm{a_{left}}}{2}} \tag{3.119}$$

$$\mathrm{Cc} = \frac{\varphi \cdot \mathrm{Mpr^{-}}_{\mathrm{right}}}{\mathrm{h_{right}} - \frac{\mathrm{a_{right}}}{2}} \tag{3.120}$$

$$\mathrm{Vjh} = \mathrm{Ts} + \mathrm{Cc} - \mathrm{V_{column}} \tag{3.121}$$

with:

Aj = cross-section area

Vn = maximum nominal shear

V_{column} = column shear strength

Vjh = horizontal joint shear force

The shear requirement is for the value of Vjh to be less than Vn (Vjh < Vn) for the column dimension to be considered safe. Furthermore, the joint shear stress analysis is also carried out.

$$\tau jh = \frac{Vjh}{Aj} \tag{3.122}$$

$$\tau jh_{max} = 1.7 \cdot \sqrt{f'c} \tag{3.123}$$

The shear stress requirement is for the value of τjh to be less than the maximum value of τjh (τjh < τjh max) for the column dimension to be considered safe.

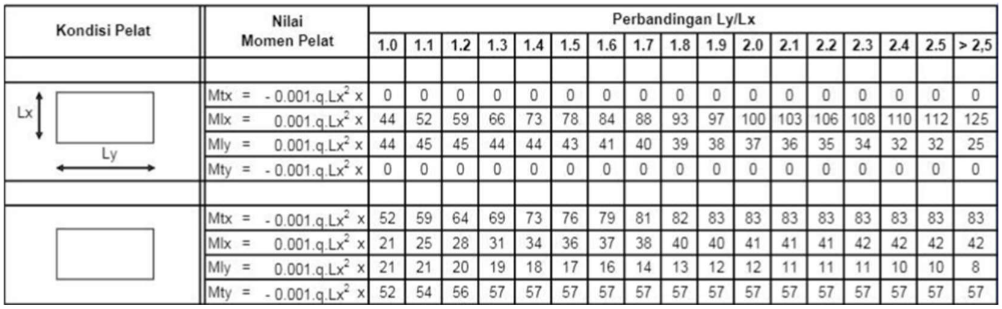
3.8.4 Floor Plate Reinforcement Design

The initial data needed for the floor plate are as follows.

f'c = compressive strength of concrete
fy_2 = tensile strength of reinforcing steel (reinforcement diameter < 12 mm)
fy_1 = tensile strength of reinforcing steel (reinforcement diameter > 12 mm)
$D_{flexural}$ = diameter of flexural reinforcement
D_{shear} = diameter of shear reinforcement (stirrup)
h_{plate} = plate thickness
Sb = thickness of concrete cover
Ly = length of the side in y direction
Lx = length of the side in x direction
Lny = net length of the side in y direction
Lnx = net length of the side in x direction
Qd = dead load of floor plate
Ql = live load of floor plate
Qu = ultimate load of floor plate (1.2Qd + 1.6Ql)
Mtx^- = support moment (negative) in x direction
Mlx^+ = middle span moment (positive) in x direction
Mty^- = support moment (negative) in y direction
Mly^+ = middle span moment (positive) in y direction

To calculate the moments of either the support or middle span area, the coefficient values are needed. These coefficients are obtained from the following table from PBI 1971 for four-sided plate moments due to uniform load under free and fully hinged support conditions. They can be determined according to the Lx/Ly ratio.

Table 3.16 Four-Sided Plate Moments due to Uniform Load under Free and Fully Hinged Support Conditions



Kondisi Pelat	Nilai Momen Pelat	Perbandingan Ly/Lx																
		1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0	2.1	2.2	2.3	2.4	2.5	> 2,5
Lx Ly	Mtx = - 0.001.q.Lx^2 x	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	Mlx = 0.001.q.Lx^2 x	44	52	59	66	73	78	84	88	93	97	100	103	106	108	110	112	125
	Mly = 0.001.q.Lx^2 x	44	45	45	44	44	43	41	40	39	38	37	36	35	34	32	32	25
	Mty = - 0.001.q.Lx^2 x	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	Mtx = - 0.001.q.Lx^2 x	52	59	64	69	73	76	79	81	82	83	83	83	83	83	83	83	83
	Mlx = 0.001.q.Lx^2 x	21	25	28	31	34	36	37	38	40	40	41	41	41	42	42	42	42
	Mly = 0.001.q.Lx^2 x	21	21	20	19	18	17	16	14	13	12	12	11	11	11	10	10	8
	Mty = - 0.001.q.Lx^2 x	52	54	56	57	57	57	57	57	57	57	57	57	57	57	57	57	57

(Source: PBI 1971)

The first plate condition is for when the plate is not hinged to the beams on any side, while the second plate condition is for when the plate is hinged to the beams on every side. Because the floor plate used is designed to be hinged to the beams on every side, the coefficient parameters used are the ones for the second condition.

Furthermore, the reinforcement design is divided into four types for one floor plate, which are the support and middle span areas in the X direction, as well as the support and middle span areas in the Y direction. Before designing the reinforcement, the shear forces that the plates must endure are determined.

$$\mathrm{Vu} = 0.5 \cdot 1.15 \cdot \mathrm{Qu} \cdot \mathrm{Ln} \qquad (3.124)$$

$$\mathrm{Vn} = 0.17 \cdot \sqrt{\mathrm{f'c}} \cdot \mathrm{b_w} \cdot \mathrm{d} \qquad (3.125)$$

If $\varphi\mathrm{Vn} > \mathrm{Vu}$, the dimension of the plates is considered safe and may withstand the shear forces. Furthermore, the nominal moments of the plates are determined with the following equation as a quadratic formula.

$$\mathrm{Mn} = \mathrm{Cc}\left(\mathrm{d} - \frac{\mathrm{a}}{2}\right) \qquad (3.126)$$

$$\mathrm{Mn} = \frac{\mathrm{Mu}}{\varphi} \qquad (3.127)$$

The equations used in the flexural reinforcement design of the floor plates are as follows.

$$\mathrm{As_{min}} = 0.002 \cdot \mathrm{b} \cdot \mathrm{h} \quad (3.128)$$

$$\mathrm{As_{needed}} = 0.85 \cdot \mathrm{f'c} \cdot \mathrm{a} \cdot \frac{\mathrm{b}}{\mathrm{fy}} \quad (3.129)$$

$$\mathrm{As_{balance}} = 0.85 \cdot \beta_1 \cdot \frac{\mathrm{f'c}}{\mathrm{fy}} \cdot \frac{600}{600+\mathrm{fy}} \cdot \mathrm{b} \cdot \mathrm{d} \quad (3.130)$$

$$\mathrm{As_{max}} = 0.75 \cdot \mathrm{As_{balance}} \quad (3.131)$$

$$\mathrm{s} = \mathrm{As_{1D}} \cdot \frac{\mathrm{b}}{\mathrm{As_{used}}} \quad (3.132)$$

$$\mathrm{As_{reinforcement\ used}} = \frac{\mathrm{b}}{\mathrm{s_{used}}} \cdot \mathrm{As_{1D}} \quad (3.133)$$

The spacing is then checked if the value does not surpass the value of 2d and 450 mm. If it does not surpass the value, the spacing is considered safe and may be used for flexural reinforcement.

Meanwhile, the equations used in the flexural reinforcement design of the floor plates are as follows.

$$\mathrm{As_{shear}} = 0.002 \cdot \mathrm{b} \cdot \mathrm{h} \quad (3.134)$$

As the spacing is determined using the same equation as the flexural reinforcement, the spacing is then checked if the value does not surpass the value of 5h and 450 mm. If it does not surpass the value, the spacing is considered safe and may be used for shear reinforcement.

3.8.5 Roof Plate Reinforcement Design

The reinforcement design of the roof plate is theoretically similar to the floor plate, hence the equations used have been mentioned in the previous subchapter.

3.8.6 Stairs Reinforcement Design

The reinforcement of the stairs and the stair landing are designed to withstand the ultimate moment values obtained from the ETABS model. The first analysis is carried out to design the flexural reinforcement of the stairs. The value of a is determined using the following equation as a quadratic formula.

$$\mathrm{Mn} = \mathrm{Cc}\left(\mathrm{d} - \frac{\mathrm{a}}{2}\right) \quad (3.135)$$

The area of the flexural reinforcement is determined using the following equation.

$$\mathrm{As} = \frac{0.85 \cdot \mathrm{f'c} \cdot \mathrm{a} \cdot \mathrm{B}}{\mathrm{fy}} \quad (3.136)$$

Meanwhile, the reinforcement ratio (ρ) is determined using the following equations.

$$\rho_{\text{min}} = \frac{1.4}{\text{fy}} \tag{3.137}$$

$$\rho = \frac{\text{As}}{\text{B}\cdot\text{d}} \tag{3.138}$$

With the value of the reinforcement ratio (ρ) obtained, the area (As) of the flexural reinforcement is determined using the following equation.

$$\text{As}_{\text{used}} = \rho_{\text{used}} \cdot \text{B} \cdot \text{d} \tag{3.139}$$

The flexural reinforcement spacing (s) is determined using the following equation.

$$\text{s} = \frac{\frac{1}{4}\cdot\pi\cdot\text{D}^2\cdot\text{B}}{\text{As}_{\text{used}}} \tag{3.140}$$

Meanwhile, the next analysis is carried out to design the shear reinforcement of the stairs. The area (As) and spacing (s) of the shear reinforcement are determined using the same equations with the flexural reinforcement design, with the ratio (ρ) value of 0.002.

3.9 Foundation Design

The foundation used in this design is the pile foundation type, which will be designed in a group with a pile cap.

3.9.1 Standard Penetration Test (N-SPT) Data

Soil strength is tested using a penetration test stated in N-SPT, namely the Standard Penetration Test. This data is obtained from the soil located in Pleret, Imogiri, Bantul, Yogyakarta. The soil data of N-SPT obtained at each depth can be seen in the Appendix.

3.9.2 Bearing Capacity Analysis

The P, Mx, and My values according to workload, factored gravity load and factored earthquake load are obtained from the ETABS model to analyze the bearing capacity of the piles.

According to the equation of Meyerhof for bearing capacity, the calculation of ultimate bearing capacity is calculated using the following equations.

End bearing capacity (Qp) = 40 ton/m^2 × Ap × $N_{\text{pile end}}$ (3.141)

Cover bearing capacity (Qs) = 0.2 ton/m^2 × As × N_{average} (3.142)

Ultimate bearing capacity (Qu) = Qp + Qs (3.143)

with:

Ap = End of pile area (m^2)

As = Cover of pile area (m^2)

The Safety Factor (SF) requirement is between 2.5 and 4. Hence, the allowable bearing capacity is calculated by dividing the ultimate bearing capacity with the SF value.

3.9.3 Dimension Estimation

A trial-and-error process with an initial assumption of the dimension is needed to design the pile cap. The number of piles in a pile group is estimated using the following equation.

Number of piles in a group (n_{pile}) = $P_{\max}/Q_{\text{all}}$ (3.145)

The requirement for the spacing of piles is as follows.

$$2.5D \leq S \leq 3D \quad (3.146)$$

The following figure shows the dimension parameters of the pile cap.

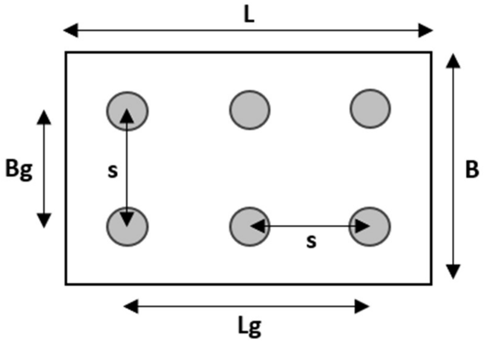


Figure 3.14 Pile Cap Dimension Parameters

The efficiency of the pile group (Eg) is calculated using the following equations.

$$\theta = \arctan\left(D/s\right) \quad (3.147)$$

$$Eg = 1 - \frac{\theta}{90} \times \left(\frac{(n-1)m + (m-1)n}{mn}\right) \quad (3.148)$$

with:

n = Number of piles in x direction

m = Number of piles in y direction

The total bearing capacity of individual piles considering the group efficiency is calculated with the following equation.

Total bearing capacity (ΣQ_{all}) $= Eg \times n \times Q_{all}$ (3.149)

To check if the total bearing capacity fits according to the requirements, the value of ΣQ_{all} must be larger than the factored gravity load ($P_{gravity}$).

Meanwhile, the weight of the pile cap ($W_{pile\ cap}$) is calculated with the following equation.

$$W_{\text{pile cap}} = \gamma_{\text{concrete}} \times L \times B \times t \tag{3.150}$$

with:

L = length dimension of the pile cap

B = width dimension of the pile cap

t = thickness or height dimension of the pile cap

The maximum and minimum axial force of a pile group are then determined to check the fulfillment of the requirement where the ultimate bearing capacity of each condition (workload, factored gravity load and factored earthquake load) has a higher value than both maximum and minimum axial forces. To determine the value of the axial forces, the maximum arm length of the pile in the x direction (x_{max}) and y direction (y_{max}) to the center of gravity must be calculated.

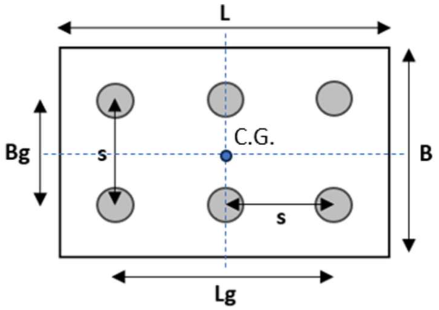


Figure 3.15 Center of Gravity Position of Pile Cap

x_{max} and y_{max} values can be obtained from the center of gravity position of pile cap.

The analysis of maximum and minimum axial forces influenced by each condition can be calculated using the following equations.

$$P_{max} = \left(\frac{P+P_{pile\ cap}}{n_{used}}\right) + \left(\frac{Mx \cdot x_{max}}{\Sigma x^2}\right) + \left(\frac{My \cdot y_{max}}{\Sigma y^2}\right) \tag{3.151}$$

$$P_{min} = \left(\frac{P+P_{pile\ cap}}{n_{used}}\right) - \left(\frac{Mx \cdot x_{max}}{\Sigma x^2}\right) - \left(\frac{My \cdot y_{max}}{\Sigma y^2}\right) \tag{3.152}$$

with:

P_{max} = Pile foundation maximum axial force (ton)

P_{min} = Pile foundation minimum axial force (ton)

3.9.4 Reinforcement Design

The ultimate and nominal moments of the pile cap are determined using the following equations.

$$Mu = \frac{n \cdot P_{max} \cdot (x_{max} - 0.5 \cdot B_{column})}{L} \tag{3.153}$$

$$Mn = \frac{Mu}{\phi_{flexural}} \tag{3.154}$$

with:

Mu = ultimate moment of pile cap

Mn = nominal moment of pile cap

ϕ = reduction factor

Furthermore, the effective height (thickness) of the pile cap is determined as follows.

$$h = t - Sb - \frac{\phi_{flexural}}{2} \tag{3.155}$$

with:

h = effective height (thickness) of the pile cap

Sb = concrete cover of the pile cap

To control the pile cap towards one-way shear, the following equation is used to determine the shear plane.

$$\text{Shear plane} = H_{column}/2 + B \tag{3.156}$$

To control the pile cap towards two-way shear, the following equations are used.

$$Vu = n(P_{max} + P_{min}) \tag{3.157}$$

$$bo = n(H_{column} + h) \tag{3.158}$$

$$\beta_{\text{column}} = \frac{B_{\text{column}}}{H_{\text{column}}} \tag{3.159}$$

$$\phi Vc1 \quad = \left(\phi\left(1 + \frac{2}{\beta_{\text{column}}}\right)\frac{\sqrt{f'c}\cdot bo\cdot h}{6}\right) \tag{3.160}$$

$$\phi Vc2 \quad = \left(\phi\left(2 + \frac{\alpha_s\cdot h}{bo}\right)\frac{\sqrt{f'c}\cdot bo\cdot h}{12}\right) \tag{3.161}$$

$$\phi Vc3 \quad = \left(\phi\left(\frac{1}{3}\right)\sqrt{f'c}\cdot bo\cdot h\right) \tag{3.162}$$

with:

Vu = factored shear force

β_{column} = ratio between the width and height dimensions of the column

ϕ = shear strength reduction factor

Vc = concrete shear strength

α_s = column location-dependent constant value

= 40 for foundations with column location in the inner building

= 30 for foundations with column location on the edge of the building

= 20 for foundations with column location on the corner edge of the building

Furthermore, the pile cap flexural and shear reinforcements are designed using the following equations.

The minimum reinforcement ratio (ρ_{min}) for the flexural reinforcement design is determined with the following equation.

$$\rho_{\text{min}} = \frac{1.4}{fy} \tag{3.163}$$

The required area and spacing of both flexural and shear reinforcements are determined with the following equations.

$$As_{\text{flexural}} = \rho_{\text{min}}\cdot B\cdot h \tag{3.164}$$

$$s = \frac{B\cdot A_{\text{flexural}}}{As_{\text{flexural}}} \tag{3.165}$$

Meanwhile, the bored pile flexural and shear reinforcements are designed using the following equations.

The required area and the minimum number of flexural reinforcements are determined with the following equations.

$$As_{\text{flexural}} = \rho_{\text{min}}\cdot Ap \tag{3.166}$$

$$n_{\text{min}} = \frac{As_{\text{flexural}}}{A_{\text{flexural}}} \tag{3.167}$$

In the meantime, the bored pile shear reinforcement requires a certain value of spacing (s) to not surpass the value of the requirements, which are listed as follows.

1. h/2
2. $16D_{flexural}$
3. $48D_{shear}$
4. Minimum length of pile cap dimension

3.10 Flexible Foundation

Foundations or footings may vibrate in any or all the six possible modes:

Mode 1: translation in the lateral direction (x)
Mode 2: translation in the longitudinal direction (y)
Mode 3: translation in the vertical direction (z)
Mode 4: rotation about the lateral axis (pitching)
Mode 5: rotation about the longitudinal axis (rocking)
Mode 6: rotation about the vertical axis (torsion or yawing)

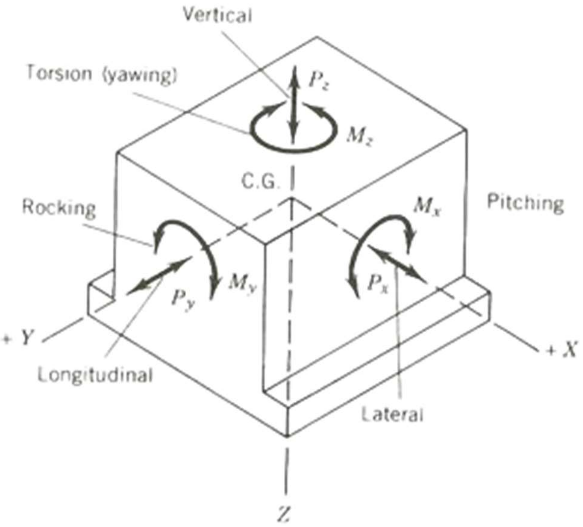


Figure 3.16 Six Modes of Foundation Vibration

(Source: Bhandari & Sengupta, 2014)

A simplified layout of footing subjected to a rocking excitation due to dynamic moment is shown in Figure 3(a). The parameters for the vibration of the foundation can be assessed by modeling the soil as a system consisting of one spring and a dashpot which supports the foundation as shown in Figure 3(b) and is commonly referred to as a vibrating system.

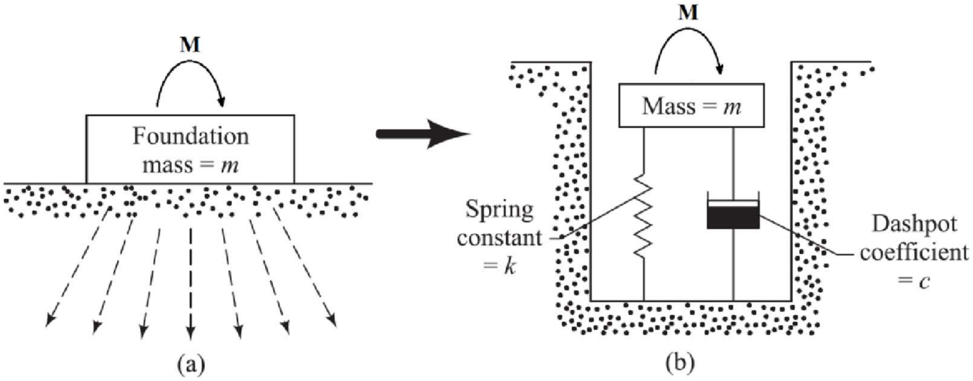


Figure 3.17 Vibrating System under Dynamic Moment

(Source: Khalesi, Mashhad, & Ahmadi, 2018)

The three vibration modes working on both fixed and flexible foundations are vertical, horizontal, and rocking. Foundation flexibility presents dynamic stiffness and dynamic damping in each vibration mode. However, due to the limitation of this research, only the dynamic spring stiffness will be studied.

3.10.1 Vertical Vibration

The equation of stiffness constant k_w^1 of one pile in a vertical direction is as follows.

$$k_w^1 = \frac{E_p \cdot A}{r_0} f_w^1 \tag{3.168}$$

with:

k_w^1 = stiffness constant of one pile in vertical direction

E_p = modulus of elasticity of pile material; Young’s modulus of pile

A = area of cross-section of H-pile section

r_0 = effective radius of one pile, equivalent radius; radius of the pile

f_w^1 = vertical stiffness parameter of a single pile

The value of f_w^1 is obtained from the following graphs.

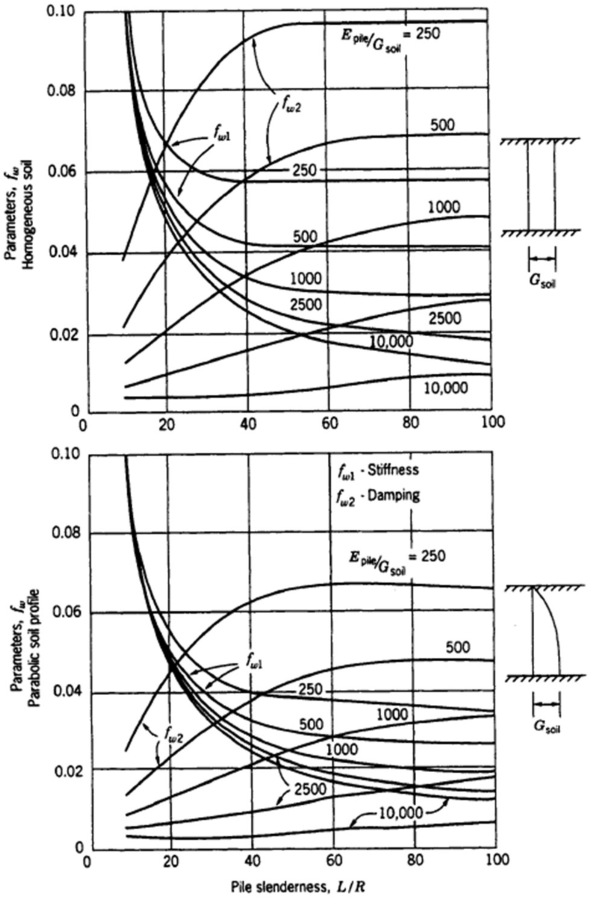


Figure 3.18 Stiffness Factors for Fixed Tip Vertically Vibrating Piles

(Source: Novak & El Sharnouby, 1983)

To analyze pile group effect, any pile in the group must be assumed as the reference pile. With the reference pile, the value of α_A, which is the axial displacement interaction factor for a typical reference pile in a group, as a function of pile length and spacing can be obtained from the following graph.

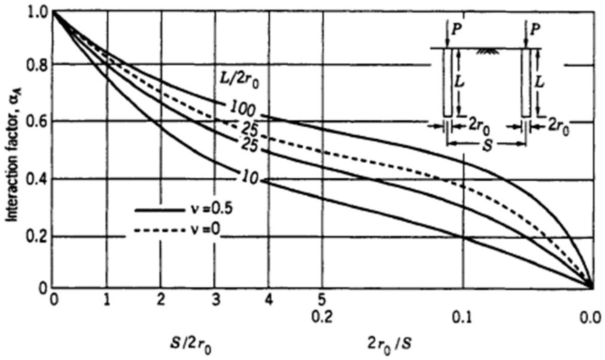


Figure 3.19 α_A as a Function of Pile Length and Spacing

(Source: Poulos, 1968)

The stiffness of a pile group requires the value of the combined stiffness of piles divided by the combined function of pile length and spacing (α_A). However, the value of pile cap spring stiffness due to side friction must be considered as well. The equations of stiffness constant k_w of pile group in the vertical direction are as follows.

$$\text{Total } k_w = k_w^g + k_w^f \tag{3.169}$$

where

$$k_w^g = \frac{\Sigma k_w^1}{\Sigma \alpha_A} \tag{3.170}$$

$$k_w^f = G_s \cdot h \cdot \overline{S_1} \tag{3.171}$$

with:

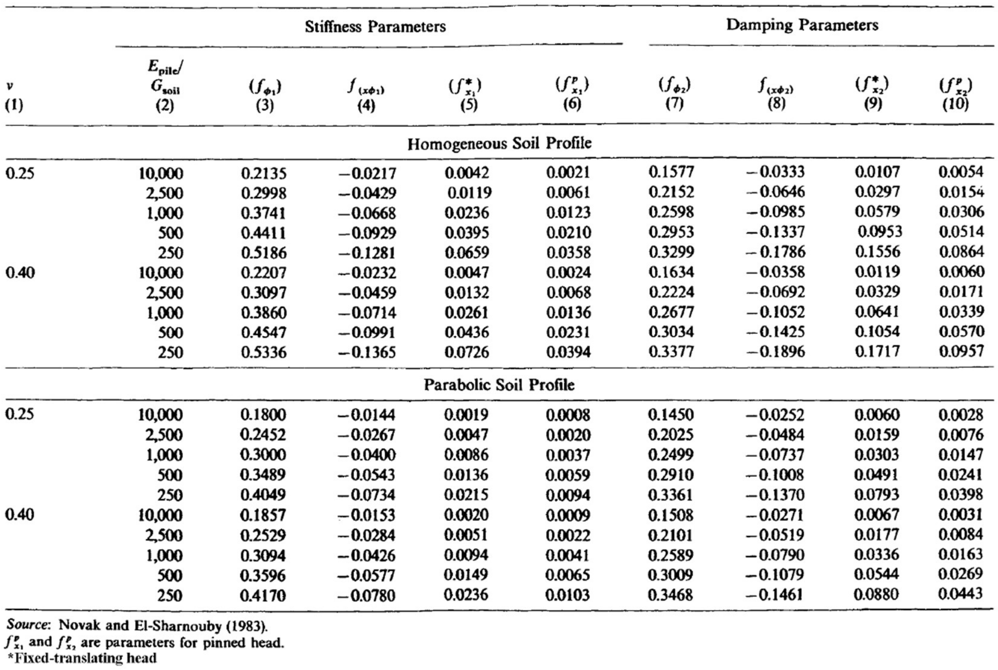
k_w^g = stiffness constant of pile group in vertical direction
k_w^f = stiffness constant of pile cap in vertical direction
G_s = shear modulus of the soil on the sides of the pile
h = depth of embedment; length of pile above ground
$\overline{S_1}$ = frequency-independent parameter of side layer for vertical vibration

With the stiffness and mass established, the response of the pile group can be determined from the principle of mechanical vibration (Prakash & Puri, 1988).

3.10.2 Lateral Vibration

In lateral or horizontal translation, the parameters of horizontal response for piles with $L/r_0 > 25$ for homogenous soil profile are obtained from the following table.

Table 3.17 Stiffness Parameters of Horizontal Response for Piles with L/r0 > 25 for Homogeneous Soil Profile and L/r0 > 30 for Parabolic Soil Profile



		Stiffness Parameters				Damping Parameters			
ν (1)	E_{pile}/G_{soil} (2)	(f_{ϕ_1}) (3)	$f_{(x\phi_1)}$ (4)	$(f^*_{x_1})$ (5)	$(f^p_{x_1})$ (6)	(f_{ϕ_2}) (7)	$f_{(x\phi_2)}$ (8)	$(f^*_{x_2})$ (9)	$(f^p_{x_2})$ (10)
				Homogeneous Soil Profile					
0.25	10,000	0.2135	−0.0217	0.0042	0.0021	0.1577	−0.0333	0.0107	0.0054
	2,500	0.2998	−0.0429	0.0119	0.0061	0.2152	−0.0646	0.0297	0.0154
	1,000	0.3741	−0.0668	0.0236	0.0123	0.2598	−0.0985	0.0579	0.0306
	500	0.4411	−0.0929	0.0395	0.0210	0.2953	−0.1337	0.0953	0.0514
	250	0.5186	−0.1281	0.0659	0.0358	0.3299	−0.1786	0.1556	0.0864
0.40	10,000	0.2207	−0.0232	0.0047	0.0024	0.1634	−0.0358	0.0119	0.0060
	2,500	0.3097	−0.0459	0.0132	0.0068	0.2224	−0.0692	0.0329	0.0171
	1,000	0.3860	−0.0714	0.0261	0.0136	0.2677	−0.1052	0.0641	0.0339
	500	0.4547	−0.0991	0.0436	0.0231	0.3034	−0.1425	0.1054	0.0570
	250	0.5336	−0.1365	0.0726	0.0394	0.3377	−0.1896	0.1717	0.0957
				Parabolic Soil Profile					
0.25	10,000	0.1800	−0.0144	0.0019	0.0008	0.1450	−0.0252	0.0060	0.0028
	2,500	0.2452	−0.0267	0.0047	0.0020	0.2025	−0.0484	0.0159	0.0076
	1,000	0.3000	−0.0400	0.0086	0.0037	0.2499	−0.0737	0.0303	0.0147
	500	0.3489	−0.0543	0.0136	0.0059	0.2910	−0.1008	0.0491	0.0241
	250	0.4049	−0.0734	0.0215	0.0094	0.3361	−0.1370	0.0793	0.0398
0.40	10,000	0.1857	−0.0153	0.0020	0.0009	0.1508	−0.0271	0.0067	0.0031
	2,500	0.2529	−0.0284	0.0051	0.0022	0.2101	−0.0519	0.0177	0.0084
	1,000	0.3094	−0.0426	0.0094	0.0041	0.2589	−0.0790	0.0336	0.0163
	500	0.3596	−0.0577	0.0149	0.0065	0.3009	−0.1079	0.0544	0.0269
	250	0.4170	−0.0780	0.0236	0.0103	0.3468	−0.1461	0.0880	0.0443

Source: Novak and El-Sharnouby (1983).
$f^p_{x_1}$ and $f^p_{x_2}$ are parameters for pinned head.
*Fixed-translating head

(Source: Novak & El Sharnouby, 1983)

With the value of f_x^1 obtained, the following shows the equations of horizontal translation stiffness constant k_x^1.

$$k_x^1 = \frac{E_p \cdot I_p}{r_0^3}(f_x^1) \qquad (3.172)$$

with:

k_x^1 = spring constant of single pile in translation

I_p = moment of inertia of pile

f_x^1 = horizontal (sliding) stiffness parameter of a free head pile

To analyze horizontal translation, the values of α_L, which is the lateral displacement interaction factor for a typical reference pile in a group, are needed. They can be obtained from the following graph.

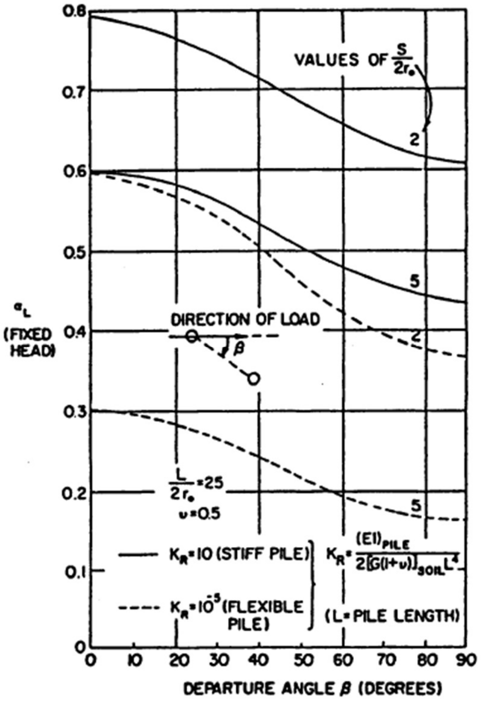


Figure 3.20 Graphical Solution for α_L

(Source: Poulos & Davis, 1972)

The stiffness of a pile group requires the value of the combined stiffness of piles divided by the combined value of α_L. However, the value of pile cap spring stiffness due to side friction must be considered as well. The equations of stiffness constant k_x of pile group in horizontal translation are as follows.

$$\text{Total } k_x = k_x^g + k_x^f \tag{3.173}$$

where

$$k_x^g = \frac{\Sigma k_x^1}{\Sigma \alpha_L} \tag{3.174}$$

$$k_x^f = G_s \cdot h \cdot \overline{S_{x1}} \tag{3.175}$$

with:

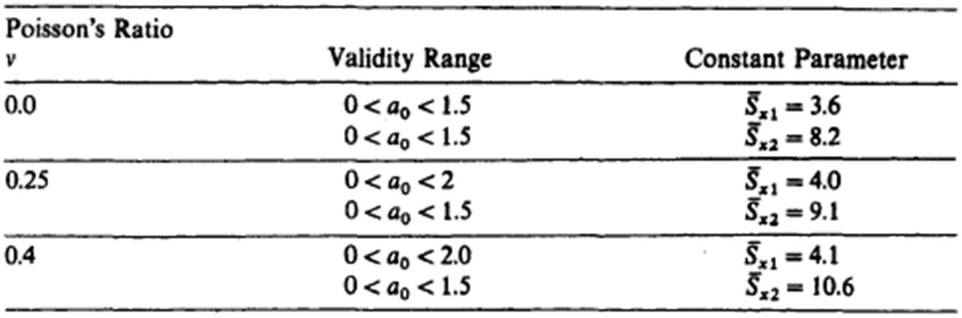
k_x^g = stiffness constant of pile group in translation

k_x^f = stiffness constant of pile cap in translation

$\overline{S_{x1}}$ = frequency-independent parameter of side layer for horizontal sliding

The value of $\overline{S_{x1}}$ is obtained from the following figure.

Table 3.18 Stiffness Constants for Half-Space and Side Layers for Sliding Vibrations



Poisson's Ratio ν	Validity Range	Constant Parameter
0.0	$0 < a_0 < 1.5$	$\bar{S}_{x1} = 3.6$
	$0 < a_0 < 1.5$	$\bar{S}_{x2} = 8.2$
0.25	$0 < a_0 < 2$	$\bar{S}_{x1} = 4.0$
	$0 < a_0 < 1.5$	$\bar{S}_{x2} = 9.1$
0.4	$0 < a_0 < 2.0$	$\bar{S}_{x1} = 4.1$
	$0 < a_0 < 1.5$	$\bar{S}_{x2} = 10.6$

(Source: Beredugo & Novak, 1972)

3.10.3 Rocking Vibration

In rocking vibration analysis, the stiffness of a single pile in both rocking alone as well as in coupled motion are calculated. With the value of f_{ϕ}^{1} obtained from Table 3.17, the following shows the equations of rocking stiffness constant k_{ϕ}^{1}.

$$k_{\phi}^{1} = \frac{E_p \cdot I_p}{r_0}\left(f_{\phi}^{1}\right) \tag{3.176}$$

with:

k_{ϕ}^{1} = stiffness constant of single pile in rocking

f_{ϕ}^{1} = rocking stiffness parameter of a pile

Meanwhile, with the value of $f_{x\phi}^{1}$ obtained from Table 3.17, the following shows the equations of cross-coupled rocking stiffness constant $k_{x\phi}^{1}$.

$$k_{x\phi}^{1} = \frac{E_p \cdot I_p}{r_0^2}\left(f_{x\phi}^{1}\right) \tag{3.177}$$

with:

$k_{x\phi}^{1}$ = cross spring stiffness of single pile

$f_{x\phi}^{1}$ = cross stiffness parameter

With the value of rocking stiffness of a single pile obtained, the value for the pile group is analyzed as follows.

$$k_{\phi}^{g} = \sum_{1}^{n}\left(k_{\phi}^{1} + k_{w}^{1} \cdot {x_r}^2 + k_{x}^{1} \cdot {z_c}^2 - 2 \cdot z_c \cdot k_{x\phi}^{1}\right) \tag{3.178}$$

with:

k_ϕ^g = stiffness constant of pile group in rocking

x_r = distance of each pile from the C.G. (center of gravity)

z_c = height of center of gravity of the pile cap above its base

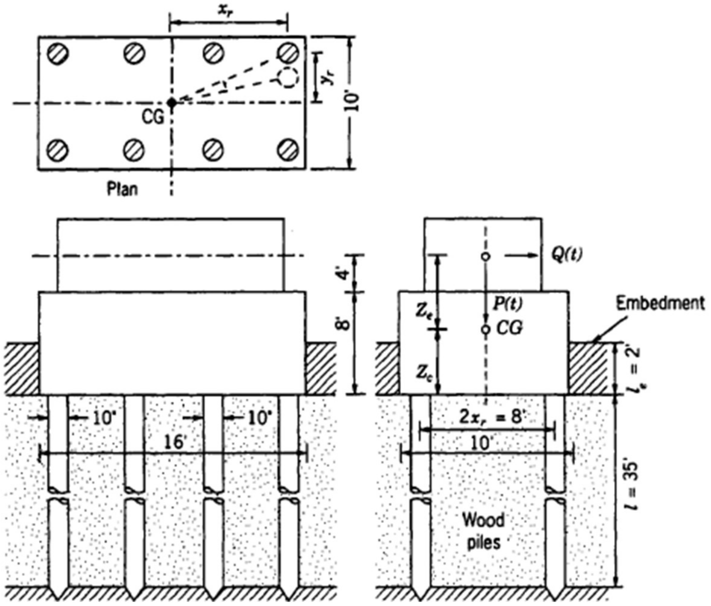


Figure 3.21 Dimensions of Pile Foundation

(Source: Prakash & Sharma, 1990)

The rocking stiffness due to the pile cap is as follows.

$$k_\phi^f = G_s \cdot r_0^2 \cdot h \cdot \overline{S_{\phi1}} + G_s \cdot r_0^2 \cdot h\left[\left(\frac{\delta^2}{3}\right) + \left(\frac{z_c}{r_0}\right)^2 - \delta\left(\frac{z_c}{r_0}\right)\right]\overline{S_{x1}} \qquad (3.179)$$

where

$$\delta = \frac{h}{r_0} \qquad (3.180)$$

with:

k_ϕ^f = stiffness constant of pile cap in rocking

$\overline{S_{\phi1}}$ = frequency-independent side layer parameter for torsional vibration

δ = angle of friction between soil and pile

The equation of the total rocking stiffness constant k_ϕ of the pile group are as follows.

$$\text{Total } k_\phi = k_\phi^g + k_\phi^f \qquad (3.181)$$

Once the stiffness of the spring is figured, its response can be established from principles of elementary mechanical vibrations (Prakash & Puri, 1988).

3.11 Fundamental Period

The fundamental period T (s) is analyzed and compared between the designed buildings with fixed and flexible support. This is to confirm the effect of having springs in the flexible foundation that changes the fundamental period of the building as well. Additionally, the building is also analyzed using pin support as a comparison, as most building designs are usually initially modelled using pin support.

To confirm the values of the fundamental period, the flexible support model with 0 stiffness values must have approximately the same fundamental period as pin support. Meanwhile, the flexible support model with infinity stiffness values must have approximately the same fundamental period as fixed support model. To test this, new models with 0 and infinity stiffness values are analyzed to determine the fundamental period values.

3.12 Internal Forces

Internal forces analysis used in structural design should always consider whether structural safety is ensured. The internal forces considered in this analysis include the shear force and flexural moment of beams and columns, drift ratio, joint rotation, and horizontal joint displacement.

3.13 Comparison Analysis

The analysis of internal forces of fixed base is compared with flexible base to figure out the effect of using flexible base instead of a fixed one. The parameters considered in the comparison analysis are as follows.

1. Fundamental period T
2. Response spectrum analysis base shear force
3. Maximum beam shear force
4. Maximum column shear force
5. Maximum beam bending moment

6. Maximum column bending moment
7. Drift ratio
8. Joint rotation
9. Horizontal joint displacement

CHAPTER IV
METHODOLOGY

4.1 Research Methodology

The research methodology used in this final project is quantitative research. Quantitative research investigates occurrences in the field through quantitative data collection and using scientific, numerical, and computer-aided tools to calculate them. The equipment used in this research is computer software such as Ms. Excel and ETABS. The research will be done during the Even Semester of 2022/2023 academic year of Universitas Islam Indonesia in Yogyakarta.

In this research, the superstructure building model is located in Pleret, Imogiri, Bantul, Yogyakarta on top of medium soil. The building itself is designed as a 15-story office building. The height of each story is identical, which is 4 meters. This makes the total height of the building 60 meters. The area of one story is 2160 m^2 without taking into account the area of the voids. With the voids considered, the area of one story is 2100 m^2. The 15-story superstructure will be exposed to external dynamic earthquake loads. After conducting structural analysis with Equivalent Lateral Force (ELF) and Response Spectrum Analysis (RSA) methods, it is also important to check the horizontal irregularity of the building, drift ratio, as well as P-delta effect. The ELF structural analysis in the first and second stages is then compared to check if the results are within the allowable drift.

Assuming the structural analysis uses a fixed / rigid foundation, the building model will show a certain fundamental period of the building after the program has been run. With the same building data, the pile foundation is designed along with the pile cap with the N-SPT (Standard Penetration Test) data obtained from the Soil Mechanics Laboratory of the Islamic University of Indonesia. Then, springs are added to the foundation in each x, y, and z direction to create a flexible support. The results will generate a difference in internal forces working on fixed and flexible foundations. Then, the structural building analysis between fixed and flexible foundations is compared, including the internal forces working on the building under dynamic loads.

4.2 Structural Model

The structural model used in the design is a 15-story building with the following floor plan on each level.

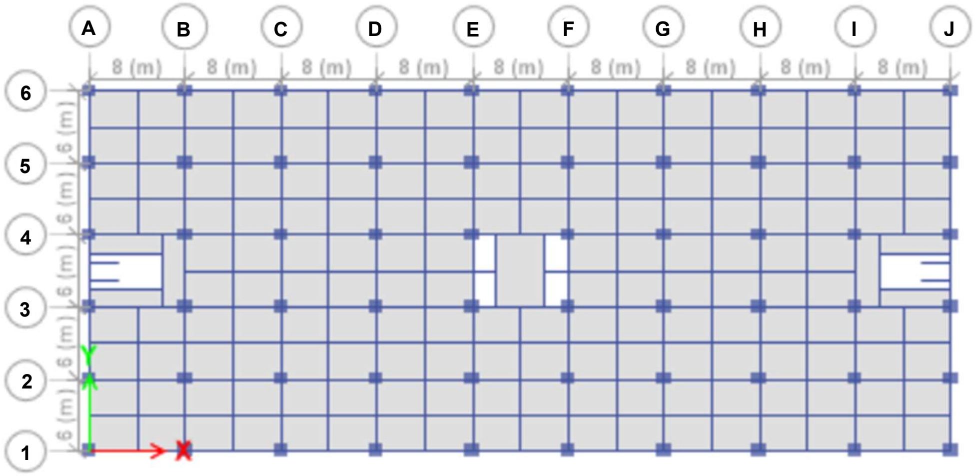


Figure 4.1 Building Floor Plan in ETABS Model

(Source: ETABS Model)

There is only one column type used in the building design, which code is C1. Meanwhile, the codification of beams used in the building is as follows.

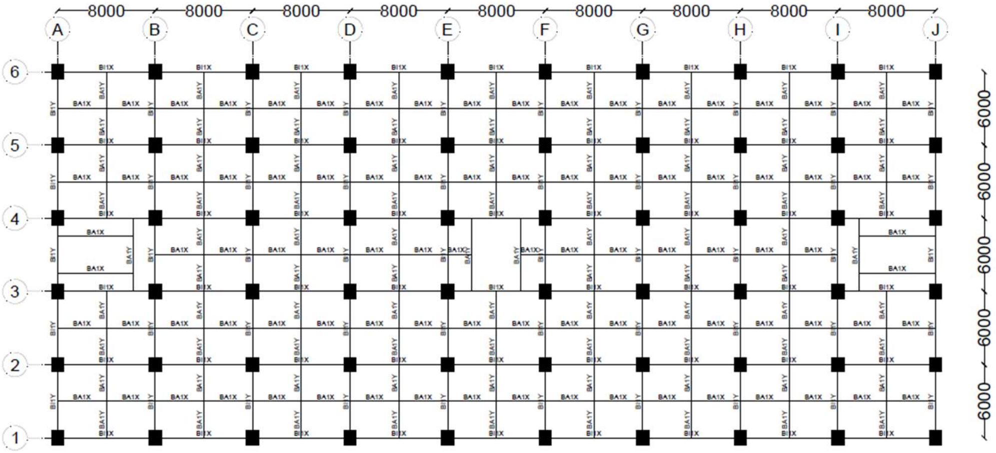


Figure 4.2 Beam Codification

(Source: AutoCAD Drawing)

Meanwhile, the front and side views of the building perimeters can be seen in the following figures.

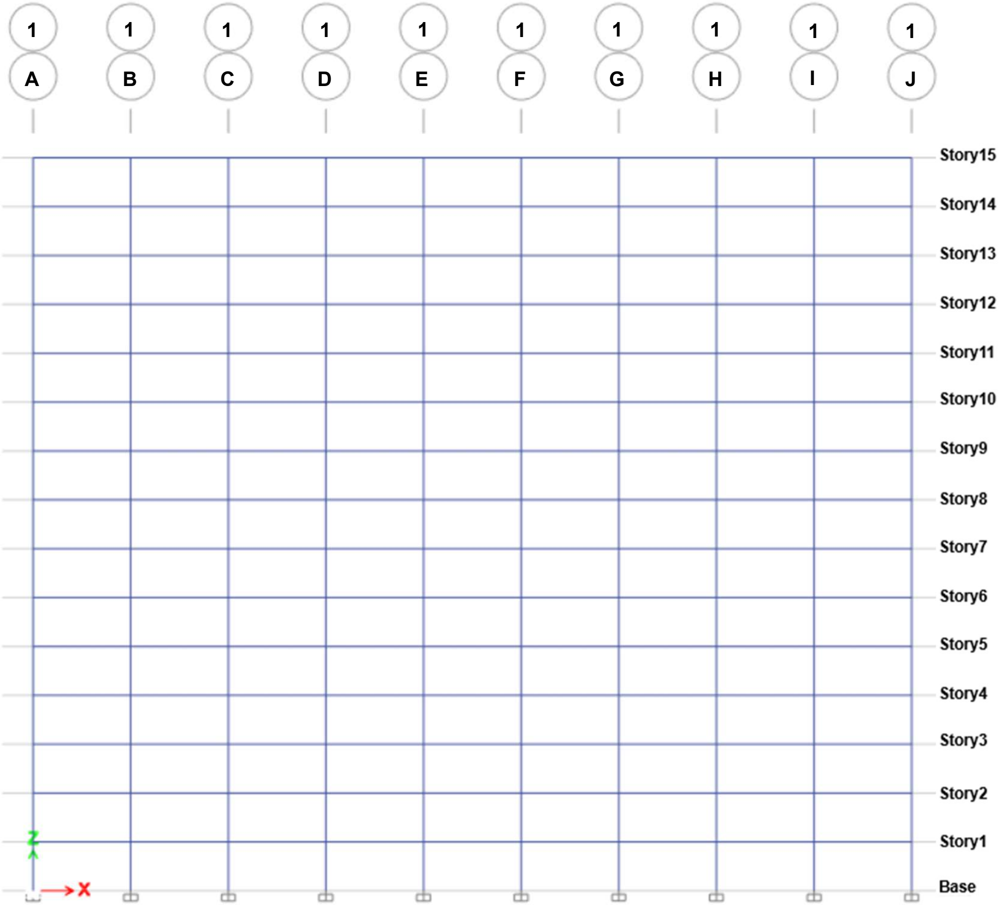


Figure 4.3 Front View of Building Perimeter in Axis 1

(Source: ETABS Model)

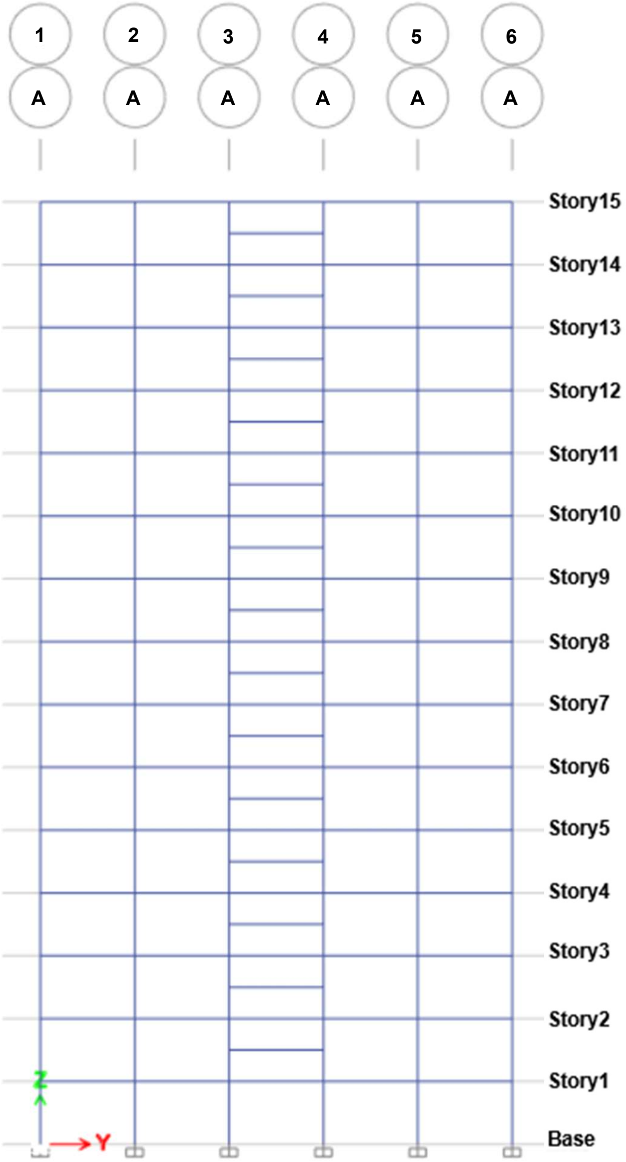


Figure 4.4 Side View of Building Perimeter in Axis A

(Source: ETABS Model)

In this step, the researcher designs the building that will further be supported by the pile foundation. This step requires analysis of the structural elements using computer software such as Ms. Excel and ETABS. The structural element inputs are such as follows.

1. Seismic design
2. Beam, column, and slab design

3. Stair design
4. Loading
5. Irregularity analysis
6. Story drift ratio and P-delta effect analysis
7. Foundation design

4.3 Method of Analysis

A building structure including all its elements can only be attained after an extensive design that is generally divided into two stages of planning. The first structural design stage considers building location, building occupation, importance factor, response spectrum, seismic design category, and method of analysis with its parameters. After structural analysis, it is imperative to check the horizontal irregularity of the building, drift ratio, as well as P-delta effect.

In seismic design, the response spectrum analysis uses the procedure and formula as formerly written in equations 3.15 until 3.18 to establish the value of period (T) and seismic response coefficient (Cs). Then, the values of Sa used to shape the graph are obtained according to its period (T) values using the procedure and formula as formerly written in equations 3.23 until 3.26.

In the first stage, dimension estimation of the height of the main and secondary beam uses equations 3.1 and 3.2. Horizontal and vertical irregularities refer to the requirements mentioned in the tables in Subchapter 3.6.1. Meanwhile, story drift uses equations 3.27 to 3.29, and P-delta effect analysis uses equations 3.30 and 3.31.

In the second stage of structural design, the effect of vertical ground motion is checked along with redundancy factor, loading scheme, and torsion analysis, followed by a follow-up structural analysis by equivalent lateral force (ELF) and response spectrum (RS) analysis method. The load combinations used in the second stage consider accidental torsion which is shown in equations 3.32 through 3.35. When the second stage of structural analysis is complete, the base shear scaling is analyzed whilst also checking the drift ratio, P-delta effect, and finally designing the elements of the structure. The method of designing the elements is also presented. After the second stage is analyzed, the next process is to design the structural member reinforcements. The members designed are main and secondary beams, columns, floor and roof plates, and stairs. The equations used are from 3.36 until 3.140.

There are two general approaches to modelling the interrelationships between structures, their foundations, and the soil that supports them, including soil flexibility and damping. One approach is called the substructural approach where the soil is represented by springs. The springs are usually oriented vertically to capture foundation rotation, which is often the dominant contributor to the SSI effect. Often, the foundation is glued to the horizontal translation. However, horizontal springs can be used to capture the ability of the foundation to move horizontally relative to the free plane, and dampers can be included to capture the damping of the foundation.

The second approach is called the direct analysis approach, in which the soil and structures are both modelled using finite elements. The soil model extends sufficiently around and under the building to account for the site properties, while seismic waves are applied to the soil boundary and excite the soil elements which in turn stimulate the structure. With its inertial weight and other properties, the structure will in turn influence soil behavior. In current practice, the direct analysis approach is typically used for large, critical projects such as nuclear power plants or large infrastructure projects such as large bridges, tunnels, subway stations, tanks, and marine structures, and requires specialized expertise.

As this research is carried out on a 15-story building structure, the substructural approach is used. This research will also analyze the effects of stiffness as the internal forces working on the pile foundation. Equations 3.159 through 3.162 are used to analyze the stiffness properties of the pile in vertical vibration according to the procedure. Meanwhile, the analysis of stiffness and properties of the pile in lateral (horizontal) vibration uses equations 3.163 to 3.166. The rocking vibration analysis uses equations 3.167 to 3.171. After obtaining the final design product, the last step is to compare the internal forces between fixed and flexible foundations under dynamic loads.

4.4 Flowchart

The following flowchart shows the stages of the final project research.

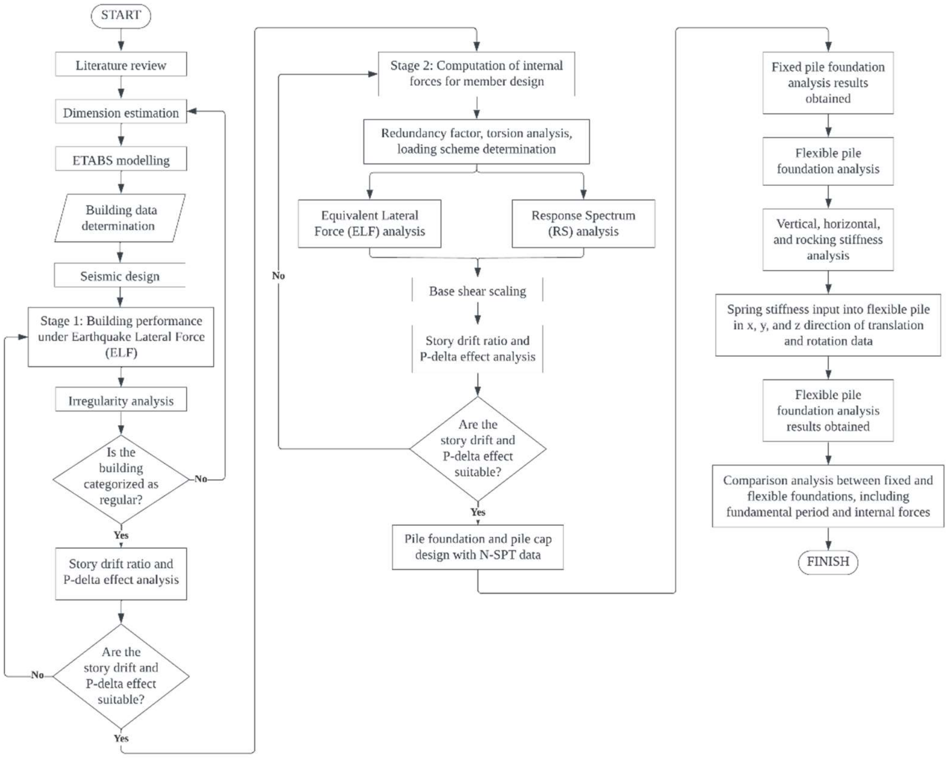


Figure 4.5 Flowchart of Final Project Research

Meanwhile, the following flowchart shows the design process of the building structure and foundation analysis using the ETABS software program.

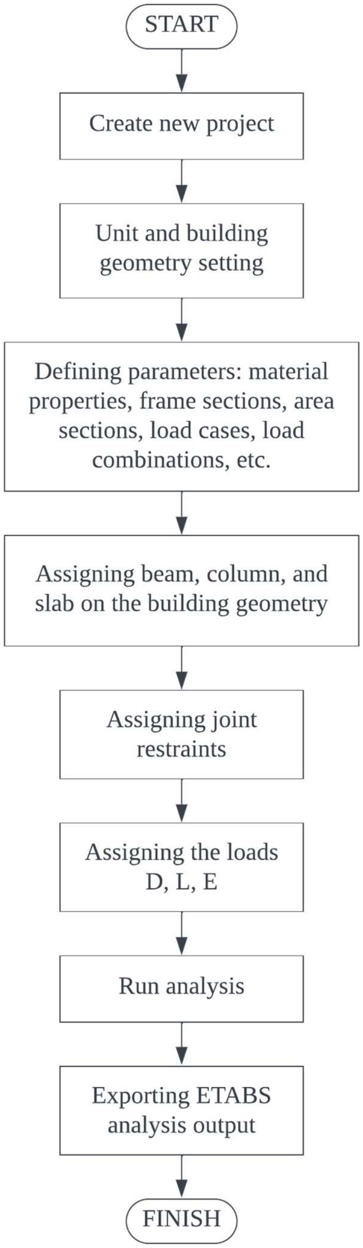


Figure 4.6 Flowchart of ETABS Program Modelling

CHAPTER V
RESULT AND DISCUSSION

5.1 Dimension Estimation

5.1.1 Main Beam Dimension Estimation

The codification of beams used in the model design can be seen in Chapter 4. The following calculation serves as an example of the calculation of the estimated dimensions of the main beams according to Equation 3.1.

1. Main beam in X direction (BI1X)

$L_{beam} = 8000 \text{ mm}$

$H_{main\ beam} = \frac{1}{10} \cdot L_{beam}$

$H_{main\ beam} = \frac{1}{10} \cdot 8000$

$H_{main\ beam} = 800 \text{ mm}$

However, after further consideration, the height of beam used in the final estimation is 900 mm.

2. Main beam in Y direction (BI1Y)

$L_{beam} = 6000 \text{ mm}$

$H_{main\ beam} = \frac{1}{10} \cdot L_{beam}$

$H_{main\ beam} = \frac{1}{10} \cdot 6000$

$H_{main\ beam} = 600 \text{ mm}$

However, after further consideration, the height of beam used in the final estimation is 900 mm.

After estimating the height of the beam, the width (B) is estimated as more or less half the height. The final estimated dimension of the main beams is as follows.

Table 5.1 Estimated Dimension of Main Beams

Codification	Length	H	B	H used	B used
BI1X	8000	800	400	900	450
BI1Y	6000	600	300	900	450

From the table above, it is concluded that both main beams BI1X and BI1Y have the same dimension, with a height of 900 mm and width of 450 mm.

5.1.2 Secondary Beam Dimension Estimation

The following calculation serves as an example of the calculation of estimated dimensions of the secondary beams according to Equation 3.2.

1. Secondary beam in X direction (BA1X)

$L_{\text{beam}} = 8000 \text{ mm}$

$H_{\text{secondary beam}} = \frac{1}{12} \cdot L_{\text{beam}}$

$H_{\text{secondary beam}} = \frac{1}{12} \cdot 8000$

$H_{\text{secondary beam}} = 666.67 \text{ mm}$

However, after further consideration, the height of beam used in the final estimation is 700 mm.

2. Secondary beam in Y direction (BA1Y)

$L_{\text{beam}} = 6000 \text{ mm}$

$H_{\text{secondary beam}} = \frac{1}{12} \cdot L_{\text{beam}}$

$H_{\text{secondary beam}} = \frac{1}{12} \cdot 6000$

$H_{\text{secondary beam}} = 500 \text{ mm}$

After estimating the height of the beam, the width (B) is estimated as more or less half the height. The final estimated dimension of the secondary beams is as follows.

Table 5.2 Estimated Dimension of Secondary Beams

Codification	Length	H	B	H used	B used
BA1X	8000	666.67	333.33	700	350
BA1Y	6000	500	250	500	250

From the table above, it is concluded that secondary beam BA1X has a height of 700 mm and width of 350 mm, while beam BA1Y has a height of 500 mm and width of 250 mm.

5.1.3 Column Dimension Estimation

All the columns used in the model are designed with the same dimension and codification. The column orientation used in this study has been previously discussed in the author's paper for the Proceeding of Civil Engineering, Environmental, Disaster & Risk Management Symposium (CEEDRiMS) 2023, titled "*Pengaruh Orientasi Kolom Terhadap Ketidakberaturan Horisontal Bangunan*". The paper discusses the influence of column orientation on the horizontal irregularities of a building so that it can provide a good, efficient, and optimal contribution to the strength of the building structure. The building configuration analyzed in the paper is the same as the one used in this study, with four types of column orientation analyzed to withstand the 15-story building. The results show that even though the structural plan is symmetrical in both directions, the orientation of the columns has a significant influence on the presence or absence of horizontal irregularities in a building, especially torsional irregularities. Of the four column orientations analyzed, three of them cause torsional irregularities in the building on some of the lower stories. The one column configuration that does not cause torsional irregularities is used as the column orientation in this study. This column orientation posseses the following dimensions.

$H = 120 \text{ cm} = 1.2 \text{ m}$

$B = 100 \text{ cm} = 1 \text{ m}$

$A_{column} = B \times H = 1 \times 1.2 = 1.2 \text{ m}^2$

5.1.4 Floor Plate Dimension Estimation

Based on the size of the building, the floor plate is first estimated to have a hypothetical thickness of 140 mm.

According to the floor plan of the building, every floor plate/slab has the same length dimension. The dimensions are as follows.

$Ly = 3000 \text{ mm} = 3 \text{ m}$

$Lx = 4000 \text{ mm} = 4 \text{ m}$

According to SNI 2847:2019, if the value of Ly/Lx is less than 2, the slab is considered two-way. On the other hand, if it is equal to or exceeds the value of 2, the slab is considered one-way. The calculation for the floor plate used in this design is as follows.

Ly/Lx = 3000/4000 = 0.75 < 2

Hence, the floor plate is considered two-way. Based on Table 3.1, the minimum thickness (h) of the slab is calculated according to the ratio of the bending stiffness of the beam section to the bending stiffness of the plate width limited laterally by the center line of the adjacent panel (if any) on each side of the beam (α_f). An example of the calculation of α_{f1} is as follows.

The floor slab used as an example is surrounded by 4 types of beams on each side. BI1X and BA1X with a length of 4000 mm along with BI1Y and BA1Y with a length of 3000 mm. α_{f1} is the ratio for BI1X, α_{f2} for BI1Y, α_{f3} for BA1X, α_{f4} for BA1Y.

$$\text{Ecb} = 4700\sqrt{f'c} = 4700\sqrt{35} = 27805.57 \text{ MPa}$$

$$\text{Ecs} = 4700\sqrt{f'c} = 4700\sqrt{35} = 27805.57 \text{ MPa}$$

$$\text{Ib} = \frac{1}{12} \cdot \text{Hb} \cdot \text{Bb}^3 = \frac{1}{12} \cdot 900 \cdot 450^3 = 6.83 \cdot 10^9 \text{ mm}^4$$

$$\text{Is} = \frac{1}{12} \cdot \text{Ls} \cdot \text{hs}^3 = \frac{1}{12} \cdot 4000 \cdot 140^3 = 0.91 \cdot 10^9 \text{ mm}^4$$

$$\alpha_{f1} = \frac{E_{cb} \cdot I_b}{E_{cs} \cdot I_s} = \frac{27805.57 \cdot 6.83 \cdot 10^9}{27805.57 \cdot 0.91 \cdot 10^9} = 7.47$$

After calculating the value of α_{f1}, the others are calculated as well. The values are as follows.

$$\alpha_{f2} = 9.96$$

$$\alpha_{f3} = 2.73$$

$$\alpha_{f4} = 0.95$$

Furthermore, the average value of α_f for all beams at the edge of the panel is shown as follows.

α_{fm} = $(\alpha_{f1}+\alpha_{f2}+\alpha_{f3}+\alpha_{f4})/4 = (7.47+9.96+2.73+0.95)/4 = 5.28 > 2$

As a result, α_{fm} is more than 2. Hence, the minimum thickness taken is either 90 mm or calculated as follows.

$$\beta = \frac{\text{Lny}}{\text{Lnx}} = \frac{2650}{3600} = 0.74$$

$$h_{min} = \frac{Ln\left(0.8+\frac{fy}{1400}\right)}{36+9\beta} = \frac{2650\left(0.8+\frac{360}{1400}\right)}{36+9\cdot 0.74} = 65.72 \text{ mm}$$

Because 90 mm is a larger value than 65.72 mm, the minimum thickness is taken as 90 mm. However, after further consideration, the thickness of the floor plate/slab is taken as 140 mm.

5.1.5 Roof Plate Dimension Estimation

Based on the size of the building, the roof plate is first estimated to have a hypothetical thickness of 100 mm. The calculation for roof plate/slab minimum thickness is the exact same as the floor plate due to the same floor plan. Hence, the minimum thickness taken is 90 mm. However, after further consideration, the thickness of the roof plate/slab is taken as 100 mm.

5.1.6 Stairs Dimension Estimation

In the geometry design of stairs, the initial data are gathered as follows.

Story height (H) = 4 m = 400 cm
Space width (B) = 6 m = 600 cm
Stair width = 3 m = 300 cm
Estimated *optrede* (s) height = 17 cm
Estimated *antrede* (a) width = 30 cm

Furthermore, the requirement checking according to Equation 3.5 is calculated as follows.

$59 \text{ cm} \leq (2s + a) \text{ cm} \leq 65 \text{ cm}$

$59 \text{ cm} \leq (2(17) + 30) \text{ cm} \leq 65 \text{ cm}$

$59 \text{ cm} < 64 \text{ cm} < 65 \text{ cm}$

Hence, the *optrede* (s) and *antrede* (a) requirement is fulfilled. Then, the number of stairs needed (n) is calculated according to Equation 3.6 as follows.

$n = (H_{story}/s) - 1 = (400/17) - 1 \approx 23$ stairs

With 23 stairs, it means that there are 11 stairs below and 12 above the stair landing. Meanwhile, the tilt angle of the stairs is calculated according to Equation 3.7 as follows.

$\alpha = \text{arc tan } (s/a) = \text{arc tan } (17/30) = 29.54°$

Furthermore, the ideal length of stairs is obtained 3.53 m with the height of stair landing at 2 m. The stair landing length obtained 2.47 m. The estimated thickness of the stair plate/slab is 30 cm or 300 mm. Meanwhile, the equivalent thickness of stairs is calculated according to Equation 3.8 as follows.

t1 = (1/2) s × cos α = (1/2) 17 × cos 29.54 = 7.40 cm

With these data of the stairs dimensions obtained, the final geometry of stairs can be seen in the following figure in the side view, with the units in millimeters.

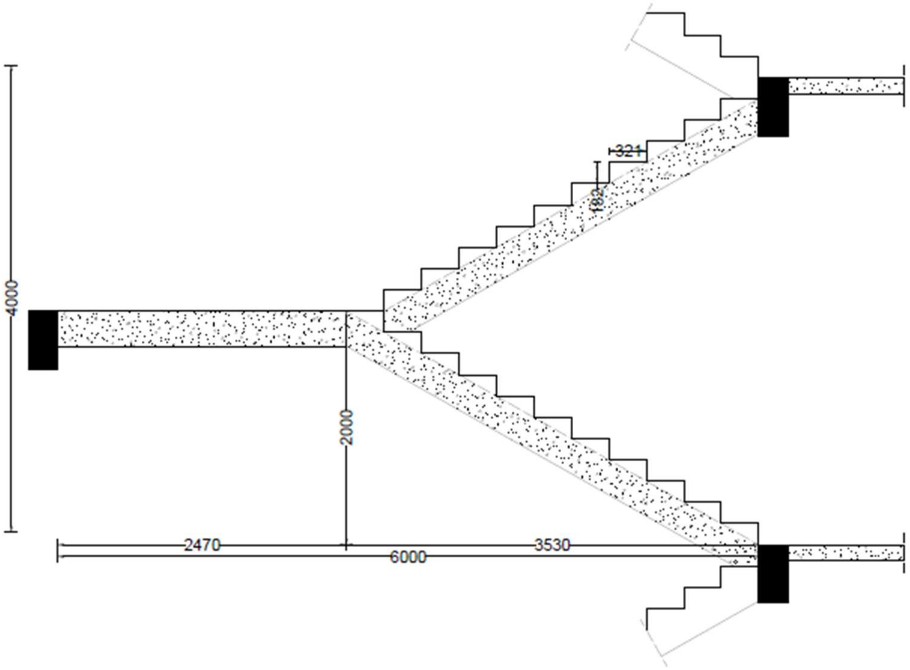


Figure 5.1 Side View of Stairs Geometry

Meanwhile, the width of the stairs is determined as 1.45 m and the width of the stair landing is 3 m.

5.2 Building Weight

Building weight (W) calculation is broken down according to each element in every story. The elements are main and secondary beams, columns, perimeter walls, floor and roof plates, as well as stairs. The calculation of each element's weight, including dead load and addtitional dead loads in one story is as follows.

5.2.1 Main Beam Weight

The element weight calculation of BI1X on one story is as follows.

$H_{BI1X} = 900\text{ mm} = 0.9\text{ m}$

$B_{BI1X} = 450\text{ mm} = 0.45\text{ m}$

$L_{BI1X} = 8000\text{ mm} = 8\text{ m}$

Number of BI1X elements on one story (n_{BI1X}) = 54

Volume $= H_{BI1X} \times B_{BI1X} \times L_{BI1X} \times n_{BI1X}$

$= 0.9 \times 0.45 \times 8 \times 54 = 174.96\text{ m}^3$

W $= \text{Volume} \times \text{Specific gravity of concrete}$

$= 174.96 \times 24 = 4199.04\text{ kN}$

Meanwhile, the element weight calculation of BI1Y on one story is as follows.

$H_{BI1Y} = 900\text{ mm} = 0.9\text{ m}$

$B_{BI1Y} = 450\text{ mm} = 0.45\text{ m}$

$L_{BI1Y} = 6000\text{ mm} = 6\text{ m}$

Number of BI1Y elements on one story (n_{BI1Y}) = 50

Volume $= H_{BI1Y} \times B_{BI1Y} \times L_{BI1Y} \times n_{BI1Y}$

$= 0.9 \times 0.45 \times 6 \times 50 = 121.50\text{ m}^3$

W $= \text{Volume} \times \text{Specific gravity of concrete}$

$= 121.50 \times 24 = 2916\text{ kN}$

Hence, the total weight of the main beams on one story is as follows.

$W_{total} = W_{BI1X} + W_{BI1Y} = 4199.04 + 2916 = 7115.04\text{ kN}$

5.2.2 Secondary Beam Weight

With $H_{BA1X} = 0.7$ m and $B_{BA1X} = 0.35$ m, the element weight of BA1X can be calculated. Using the same method as the main beams, the total volume of BA1X elements on one story is 89.18 m^3 and the weight (W) is 2140.32 kN. Meanwhile, the element weight of BA1Y can be calculated with known dimensions $H_{BA1Y} =$ 0.5 m and $B_{BA1Y} = 0.25$ m. The total volume of BA1Y elements obtained on one story is 34.5 m^3 and the weight (W) is 828 kN.

Hence, the total weight of the secondary beams on one story is as follows.

$W_{total} = W_{BA1X} + W_{BA1Y} = 2140.32 + 828 = 2968.32\text{ kN}$

Finally, the total weight of both the main and secondary beams on one story is as follows.

$\mathrm{W_{total\ beams}} = \mathrm{W_{main\ beam}} + \mathrm{W_{secondary\ beam}}$

$\mathrm{W_{total\ beams}} = 7115.04 + 2968.32 = 10083.36\ \mathrm{kN}$

5.2.3 Column Weight

The element weight calculation of C1 column on one story is as follows.

$\mathrm{H_{C1}} = 1200\ \mathrm{mm} = 1.2\ \mathrm{m}$

$\mathrm{B_{C1}} = 1000\ \mathrm{mm} = 1\ \mathrm{m}$

$\mathrm{L_{C1}} = 4000\ \mathrm{mm} = 4\ \mathrm{m}$

Number of C1 column elements on one story ($\mathrm{n_{C1}}$) = 60

Volume $= \mathrm{H_{C1}} \times \mathrm{B_{C1}} \times \mathrm{L_{C1}} \times \mathrm{n_{C1}}$

$= 1.2 \times 1 \times 4 \times 60 = 288\ \mathrm{m^3}$

W $= \text{Volume} \times \text{Specific gravity of concrete}$

$= 288 \times 24 = 6912\ \mathrm{kN}$

5.2.4 Floor Plate Weight

The element weight (dead load) calculation of the floor plate on the first story without additional dead load is as follows.

$\mathrm{A_g} = 72 \times 30 = 2160\ \mathrm{m^2}$

Void $= 0\ \mathrm{m^2}$

$\mathrm{A_{net}} = 2160 - 0 = 2160\ \mathrm{m^2}$

With the floor plate thickness (t) of 140 mm or 0.14 m, the volume and weight calculations are as follows.

Volume $= \mathrm{A_{net}} \times \mathrm{t}$

$= 2160 \times 0.14 = 302.40\ \mathrm{m^3}$

$\mathrm{W_{DEAD}} = \text{Volume} \times \text{Specific gravity of concrete}$

$= 302.40 \times 24 = 7257.60\ \mathrm{kN}$

The additional dead load calculation of the floor plate on the first story is as follows.

Table 5.3 Floor Plate Additional Dead Load Components

No	Component	Volume Weight		Thickness		Q	
		Value	Unit	Value	Unit	kg/m²	kN/m²
1	Partition					48.93	0.480
2	Sand	1600	kg/m³	0.05	m	80	0.785
3	Spec	21	kg/m²/cm thickness	3	cm	63	0.618
4	Ceramic					17	0.167
5	Mechanical & Electrical					30	0.294
6	Ceiling					9	0.088
7	Ceiling Hanger					5	0.049
Total Additional Dead Load						252.93	2.481

From the table above, it is found that the total additional dead load of floor plate components is as follows.

$Q_{ADEAD} \quad = 2.48\ kN/m^2$

$W_{ADEAD} \quad = A_{net} \times Q_{ADEAD}$

$\quad = 2160 \times 2.48 = 5359.48\ kN$

Hence, the total weight of the floor plates including the additional dead load on the first story is as follows.

$W_{first\ story} = W_{DEAD} + W_{ADEAD} = 7257.60 + 5359.48 = 12617.08\ kN$

Meanwhile, the element weight (dead load) calculation of the floor plate on other stories without additional dead load is as follows.

$A_g \quad = 72 \times 30 = 2160\ m^2$

$Void \quad = 60\ m^2$

$A_{net} \quad = 2160 - 60 = 2100\ m^2$

With the floor plate thickness (t) of 140 mm or 0.14 m, the volume and weight calculations are as follows.

$Volume \quad = A_{net} \times t$

$\quad = 2100 \times 0.14 = 294\ m^3$

$W_{DEAD} \quad = Volume \times Specific\ gravity\ of\ concrete$

$\quad = 294 \times 24 = 7056\ kN$

The additional dead load calculation of the floor plate on other stories is as follows.

$Q_{ADEAD} \quad = 2.48\ kN/m^2$

$\mathrm{W_{ADEAD}} \quad = \mathrm{A_{net}} \times \mathrm{Q_{ADEAD}}$

$= 2100 \times 2.48 = 5210.60\ \mathrm{kN}$

Hence, the total weight of the floor plates including the additional dead load on other stories is as follows.

$\mathrm{W_{other\ stories}} = \mathrm{W_{DEAD}} + \mathrm{W_{ADEAD}} = 7056 + 5210.60 = 12266.60\ \mathrm{kN}$

5.2.5 Roof Plate Weight

The element weight (dead load) calculation of the roof plate without additional dead load is as follows.

$\mathrm{A_g} \quad = 72 \times 30 = 2160\ \mathrm{m^2}$

$\mathrm{Void} \quad = 60\ \mathrm{m^2}$

$\mathrm{A_{net}} \quad = 2160 - 60 = 2100\ \mathrm{m^2}$

With the floor plate thickness (t) of 100 mm or 0.10 m, the volume and weight calculations are as follows.

$\mathrm{Volume} \quad = \mathrm{A_{net}} \times \mathrm{t}$

$= 2100 \times 0.10 = 210\ \mathrm{m^3}$

$\mathrm{W_{DEAD}} \quad = \mathrm{Volume} \times \mathrm{Specific\ gravity\ of\ concrete}$

$= 210 \times 24 = 5040\ \mathrm{kN}$

The additional dead load calculation of the roof plate is as follows.

Table 5.4 Roof Plate Additional Dead Load Components

No	Component	Volume Weight		Thickness		Q	
		Value	Unit	Value	Unit	kg/m²	kN/m²
1	Spec	21	kg/m²/cm thickness	3	cm	63	0.618
2	Mechanical & Electrical					30	0.294
3	Ceiling					9	0.088
4	Ceiling Hanger					5	0.049
5	Waterproofing	2100	kg/m³	0.02	m	42	0.412
Total Additional Dead Load						149	1.462

From the table above, it is found that the total additional dead load of roof plate components is as follows.

$\mathrm{Q_{ADEAD}} \quad = 1.46\ \mathrm{kN/m^2}$

$\mathrm{W_{ADEAD}} \quad = \mathrm{A_{net}} \times \mathrm{Q_{ADEAD}}$

$= 2100 \times 1.46 = 3069.55\ \mathrm{kN}$

Hence, the total weight of the roof plates including the additional dead load is as follows.

$\mathrm{W_{roof} = W_{DEAD} + W_{ADEAD} = 5040 + 3069.55 = 8109.55\ kN}$

5.2.6 Stairs Weight

The element weight calculation of stairs on one story includes the weight of stair landing, the stairs plate, and the stairs (stair steps). It also includes a secondary beam BA1Y that connects the stair landing to the columns. The calculation for stair landing is as follows.

$\mathrm{Ag_{stair\ landing} = 2.47 \times 3 = 7.41\ m^2}$

With the plate thickness of 300 mm or 0.3 m, the volume and weight calculations of stair landing are as follows.

Volume $\mathrm{= Ag_{stair\ landing} \times t}$

$\mathrm{= 7.41 \times 0.3 = 2.22\ m^3}$

$\mathrm{W_{DEAD}}$ $\mathrm{= Volume \times Specific\ gravity\ of\ concrete}$

$\mathrm{= 2.22 \times 24 = 53.35\ kN}$

To calculate the volume and weight of stairs plate, the length of the plate must first be calculated. The calculation is as follows.

$\mathrm{L_{stairs\ plate} = 2 \times \sqrt{2^2 + 3.53^2} = 8.11\ m}$

The calculation for the stairs plate is as follows.

$\mathrm{Ag_{stairs\ plate} = 8.11 \times 1.45 = 11.77\ m^2}$

With the plate thickness of 300 mm or 0.3 m, the volume and weight calculations of stairs plate are as follows.

Volume $\mathrm{= Ag_{stairs\ plate} \times t}$

$\mathrm{= 11.77 \times 0.3 = 3.53\ m^3}$

$\mathrm{W_{DEAD}}$ $\mathrm{= Volume \times Specific\ gravity\ of\ concrete}$

$\mathrm{= 3.53 \times 24 = 84.71\ kN}$

The calculation for the stairs (stair steps) is as follows.

$\mathrm{Ag_{stair\ steps} = 8.11 \times 1.45 = 11.77\ m^2}$

With the equivalent plate thickness of 7.40 cm or 0.074 m, the volume and weight calculations of stair steps are as follows.

Volume $= A_{g_{\text{stair steps}}} \times t$

$= 11.77 \times 0.074 = 0.87\ \text{m}^3$

W_{DEAD} $= \text{Volume} \times \text{Specific gravity of concrete}$

$= 0.87 \times 24 = 20.90\ \text{kN}$

The calculation for the secondary beam BA1Y with a width dimension (B) of 0.25 m and height dimension of 0.5 m is as follows.

$A_{g_{\text{BA1Y}}} = B \times H = 0.25 \times 0.5 = 0.125\ \text{m}^2$

With the beam length of 6 m, the volume and weight calculations of secondary beam BA1Y are as follows.

Volume $= A_{g_{\text{BA1Y}}} \times L_{\text{BA1Y}}$

$= 0.125 \times 6 = 0.75\ \text{m}^3$

W_{DEAD} $= \text{Volume} \times \text{Specific gravity of concrete}$

$= 0.75 \times 24 = 18\ \text{kN}$

Hence, the total dead load of stairs is as follows.

W_{DEAD} $= W_{\text{stair landing}} + W_{\text{stairs plate}} + W_{\text{stair steps}} + W_{\text{BA1Y}}$

$= 53.35 + 84.71 + 20.90 + 18 = 176.96\ \text{kN}$

Because there are two stairs in one story, the dead load of stairs is doubled, which makes the total dead load of stairs in one story 353.93 kN.

Meanwhile, the additional dead load calculation of the stairs is as follows.

Table 5.5 Stairs Additional Dead Load Components

No	Component	Volume Weight		Thickness		Q	
		Value	Unit	Value	Unit	kg/m²	kN/m²
1	Partition					48.93	0.480
2	Sand	1600	kg/m³	0.05	m	80	0.785
3	Spec	21	kg/m²/cm thickness	3	cm	63	0.618
4	Ceramic					17	0.167
Total Additional Dead Load						208.93	2.050

From the table above, it is found that the total additional dead load of stairs components is as follows.

Q_{ADEAD} $= 2.05\ \text{kN/m}^2$

$$\mathrm{A_{net}} = \mathrm{A_{g_{stair\ landing}}} + \mathrm{A_{g_{stairs\ plate}}} + \mathrm{A_{g_{stair\ steps}}}$$

$$\mathrm{A_{net}} = 7.41 + 11.77 + 11.77 = 30.94\ \mathrm{m^2}$$

$$\mathrm{W_{ADEAD}} = \mathrm{A_{net}} \times \mathrm{Q_{ADEAD}}$$

$$= 30.94 \times 2.05 = 63.42\ \mathrm{kN}$$

Because there are two stairs in one story, the additional dead load of stairs is doubled, which makes the total additional dead load of stairs in one story 126.84 kN.

Hence, the total weight of the roof plates including the additional dead load is as follows.

$$\mathrm{W_{stairs}} = \mathrm{W_{DEAD}} + \mathrm{W_{ADEAD}} = 353.93 + 126.84 = 480.76\ \mathrm{kN}$$

Additionally, in the building design, wall weight is considered as an additional dead load. It is noted that only 70% of the additional dead load of the wall is calculated into the total weight of the building. The calculation is as follows.

Table 5.6 Wall Additional Dead Load Components

No	Component	Volume Weight		Thickness		Q	
		Value	Unit	Value	Unit	kg/m²	kN/m²
1	Brick					450	4.415
2	Plaster	21	kg/m²/cm thickness	6	cm	126	1.236
Total Additional Dead Load						403.20	3.955

From the table above, it is found that the total additional dead load of wall components is as follows.

$$\mathrm{Q_{ADEAD}} = 3.96\ \mathrm{kN/m^2}$$

$$\mathrm{A_{net}} = \text{Building perimeter} \times \text{Wall height}$$

$$\mathrm{A_{net}} = (2 \cdot 72 + 2 \cdot 30) \times 4 = 816\ \mathrm{m^2}$$

$$\mathrm{W_{ADEAD}} = \mathrm{A_{net}} \times \mathrm{Q_{ADEAD}}$$

$$= 816 \times 3.96 = 3227.60\ \mathrm{kN}$$

Hence, the additional dead load of the wall of one story is obtained as much as 3227.60 kN.

Based on the calculations of each element’s weight on every story, the final weight of the building is calculated as follows.

1. Story 1

The first story consists of beams, columns, floor plates/slabs, stairs, and walls. The total weight of each element can be seen in the following table.

Table 5.7 Total Element Weight of Story 1

Element	Wn (kN)	
	Without ADL	With ADL
Beam	10083.36	10083.36
Column	6912	6912
Plate	7257.60	12617.08
Stairs	353.93	480.76
Wall	0	3227.60
Total	24606.89	33320.80

2. Stories 2-14

Stories 2-14 consist of beams, columns, floor plates/slabs, stairs, and walls. The total weight of each element can be seen in the following table.

Table 5.8 Total Element Weight of Stories 2-14

Element	Wn (kN)	
	Without ADL	With ADL
Beam	10083.36	10083.36
Column	6912	6912
Plate	7056	12266.60
Stairs	353.93	480.76
Wall	0	3227.60
Total	24405.29	32970.33

3. Story 15 (roof)

The top story (roof) consists of beams, columns, and roof plates/slabs. The total weight of each element can be seen in the following table.

Table 5.9 Total Element Weight of Story 15 (Roof)

Element	Wn (kN)	
	Without ADL	With ADL
Beam	10083.36	10083.36
Column	6912	6912
Plate	5040	8109.55
Stairs	0	0
Wall	0	0
Total	22035.36	25104.91

Finally, the total weight of the building can be seen in the following table.

Table 5.10 Total Building Weight

Story	Wn (kN)	
	Without ADL	With ADL
15	22035.36	25104.909
14	24405.28526	32970.32568
13	24405.28526	32970.32568
12	24405.28526	32970.32568
11	24405.28526	32970.32568
10	24405.28526	32970.32568
9	24405.28526	32970.32568
8	24405.28526	32970.32568
7	24405.28526	32970.32568
6	24405.28526	32970.32568
5	24405.28526	32970.32568
4	24405.28526	32970.32568
3	24405.28526	32970.32568
2	24405.28526	32970.32568
1	24606.89	33320.80
Total	363910.95	487039.94

Hence, the total weight of the building without the additional dead load is 363910.95 kN, while including the additional dead load is 487039.94 kN.

5.3 Seismic Design of the Structural Model with Fixed Support

5.3.1 Data Determination

1. Classification of risk category and importance factor, Ie

   The building has 15 stories built using reinforced concrete. The occupation of the building is an office. Based on the requirement by SNI 1726:2019, the risk category falls under category II. Meanwhile, the importance factor Ie can be categorized as 1.0 as required by SNI 1726:2019.

2. Classification of soil site

   The building designed in this study is located in Yogyakarta, Indonesia with medium soil (site class SD) according to SNI 1726:2019.

3. Ground motion parameters, Ss and S1

   The Ss parameter shows the risk-targeted maximum considered earthquake ground motion (MCE_R) for the Indonesian region for the response spectrum of 0.2-second (5% critical damping). The Ss value obtained from the map provided by 1726:2019 is 1.107G. Meanwhile, the S1 parameter shows the risk-targeted maximum considered earthquake ground motion (MCE_R) for the Indonesian region for the response spectrum of 1-second (5% critical damping). The S1 value obtained from the map provided by 1726:2019 is 0.507G.

4. Classification of site coefficient, Fa and Fv

   Based on the requirement by SNI 1726:2019, the value of site coefficient Fa is interpolated to 1.057. Meanwhile, the value of site coefficient Fv is interpolated to 1.793.

5. Response spectrum graph parameters

   The equations used for this calculation are Equations 3.9 to 3.14. The parameters of the response spectrum graph are calculated as follows.

   SMS = Fa×Ss = 1.170G
   SM1 = Fv×S1 = 0.909G
   SDS $= \frac{2}{3}\times$SMS = 0.780G
   SD1 $= \frac{2}{3}\times$SM1 = 0.606G
   T_0 = 0.2×(SD1/SDS) = 0.155 seconds
   T_S = 1×(SD1/SDS) = 0.777 seconds

   Meanwhile, the value of T_L or long-period transition can be seen in the map provided by 1726:2019. From the map, a value of 6 seconds is obtained for the long transition period T_L in Yogyakarta.

6. Seismic design category, based on SDS and SD1

   Based on the requirement by SNI 1726:2019, the seismic design category of the building based on SDS is obtained as D. Meanwhile, the seismic design category of the building based on SD1 is obtained as D.

7. Determination of R, Ω, and Cd values

   With the Special Moment Resisting Frame System (SRPMK) adopted as the seismic force-resisting system, the following values are obtained:

   Response modification coefficient, R = 8
   System exceeding strength factor, Ω = 3
   Deflection magnification factor, Cd = 5.5

8. Determination of approach fundamental period, Ta

   The coefficient Cu for the upper bound on the calculated period is determined based on the requirement by SNI 1726:2019. The value of Cu obtained is 1.4. The parameter values for the approach period, Ct and x are also determined based on the requirement by SNI 1726:2019. The value of Ct obtained is 0.0466 and x is 0.9. With the height of the superstructure, hn = 15×4 m = 60 m, using Equation 3.15, the approach period Ta is calculated as follows.

   Ta = Ct×hn^x

   = 0.0466×60$^{0.9}$

   = 1.86 seconds

   Meanwhile, the upper bound of the calculated period is determined as follows.

   Cu×Ta = 1.4×1.86 s = 2.60 seconds

9. Calculation of seismic response coefficient, Cs

   The equations used for this calculation are Equations 3.16, 3.17, and 3.18. Seismic response coefficient Cs must be determined as follows.

   $$Cs_1 = \frac{SDS}{\left(\frac{R}{Ie}\right)} = 0.098G$$

   For Ta ≤ T_L, it is not necessary for the Cs value to surpass the following value.

   $$Cs_2 = \frac{SD1}{Ta\left(\frac{R}{Ie}\right)} = 0.031G$$

   Finally, the Cs value must not exceed the following value.

   $$Cs_{min} = 0.044 \times SDS \times Ie \geq 0.01G$$

   $$Cs_{min} = 0.034G \geq 0.01G$$

   Hence, the Cs value used is 0.034G.

10. Seismic base shear force (V)

   Previously, the Cs value has been obtained with the value of 0.034G. Meanwhile, the building weight (W) value is 487039.94 kN. The seismic base shear force (V) is determined according to Equation 3.19. The calculation is as follows.

   $$V = C_x \times W = 0.034 \times 487039.94 = 16719.79 \text{ kN}$$

Additionally, the constant k value needed to determine the lateral seismic force (Fx) is determined according to the ETABS fundamental period value for the fixed support model. The value itself is determined with interpolation between 0.5 s and 2.5 s period value, with the constant k value of 1 for the smallest (low-rise building) and 2 for the largest (high-rise building). In this case, because the fundamental period value is 2.45 s, the constant k value is interpolated to 1.97.

11. Lateral seismic force (Fx)

    The lateral seismic force (Fx, in kN) at any story is determined with Equation 3.20. The data needed is obtained from the following table.

Table 5.11 Wx, hx, and k Values of Each Story

Story	Wx (kN)	hx (m)	k	$W_x h_x{}^k$ (kNm)
15	25104.91	60.00	1.97	80829363.33
14	32970.33	56.00	1.97	92645520.24
13	32970.33	52.00	1.97	80044734.18
12	32970.33	48.00	1.97	68352838.17
11	32970.33	44.00	1.97	57571820.54
10	32970.33	40.00	1.97	47703848.08
9	32970.33	36.00	1.97	38751300.81
8	32970.33	32.00	1.97	30716817.82
7	32970.33	28.00	1.97	23603359.84
6	32970.33	24.00	1.97	17414297.33
5	32970.33	20.00	1.97	12153540.61
4	32970.33	16.00	1.97	7825743.78
3	32970.33	12.00	1.97	4436651.93
2	32970.33	8.00	1.97	1993769.87
1	33320.80	4.00	1.97	513353.60
Total	487039.94			564556960.14

From the table above, the total $W_x h_x{}^k$ is obtained 564556960.14 kNm. The k coefficient is determined with interpolation as explained beforehand in Subchapter 3.6.1. The interpolated k value obtained is 1.97 because the 15-story building designed is considered a high-rise building. Meanwhile, according to Equation 3.21, the Cvx calculation example of the top story is as follows.

$$\mathrm{Cv}_{15} = \frac{\mathrm{W}_{15}\mathrm{h}_{15}{}^{\mathrm{k}}}{\sum_{\mathrm{i}}^{15} \mathrm{W}_{\mathrm{i}}\mathrm{h}_{\mathrm{i}}{}^{\mathrm{k}}} = \frac{80829363.33}{564556960.14} = 0.14$$

This calculation process continues until the first story. Then, the total of Cvx values on every story must be equal to 1. The final values can be seen in the following table.

Table 5.12 Cvx Values of Each Story

Story	Wx (kN)	hx (m)	k	$W_x h_x^k$ (kNm)	Cvx
15	25104.91	60.00	1.97	80829363.33	0.14
14	32970.33	56.00	1.97	92645520.24	0.16
13	32970.33	52.00	1.97	80044734.18	0.14
12	32970.33	48.00	1.97	68352838.17	0.12
11	32970.33	44.00	1.97	57571820.54	0.10
10	32970.33	40.00	1.97	47703848.08	0.08
9	32970.33	36.00	1.97	38751300.81	0.07
8	32970.33	32.00	1.97	30716817.82	0.05
7	32970.33	28.00	1.97	23603359.84	0.04
6	32970.33	24.00	1.97	17414297.33	0.03
5	32970.33	20.00	1.97	12153540.61	0.02
4	32970.33	16.00	1.97	7825743.78	0.01
3	32970.33	12.00	1.97	4436651.93	0.01
2	32970.33	8.00	1.97	1993769.87	0.004
1	33320.80	4.00	1.97	513353.60	0.001
Total	487039.94			564556960.14	1

From the table above, the total Cvx is obtained as 1, which means it fulfills the requirement. With V = 16719.79 kN, the Fx can be calculated using Equation 3.20. The calculation example of the top story is as follows.

$$F_{15} = Cv_{15} \times V = 0.14 \times 16719.79 = 2393.82 \text{ kN}$$

This calculation process continues until the first story. The final values can be seen in the following table.

Table 5.13 Fx Values of Each Story

Story	Wx (kN)	hx (m)	k	$W_x h_x^k$ (kNm)	Cvx	Fx (kN)
15	25104.91	60.00	1.97	80829363.33	0.14	2393.82
14	32970.33	56.00	1.97	92645520.24	0.16	2743.77
13	32970.33	52.00	1.97	80044734.18	0.14	2370.59
12	32970.33	48.00	1.97	68352838.17	0.12	2024.32
11	32970.33	44.00	1.97	57571820.54	0.10	1705.03
10	32970.33	40.00	1.97	47703848.08	0.08	1412.79
9	32970.33	36.00	1.97	38751300.81	0.07	1147.65
8	32970.33	32.00	1.97	30716817.82	0.05	909.70
7	32970.33	28.00	1.97	23603359.84	0.04	699.03
6	32970.33	24.00	1.97	17414297.33	0.03	515.74
5	32970.33	20.00	1.97	12153540.61	0.02	359.94

4	32970.33	16.00	1.97	7825743.78	0.01	231.77
3	32970.33	12.00	1.97	4436651.93	0.01	131.39
2	32970.33	8.00	1.97	1993769.87	0.004	59.05
1	33320.80	4.00	1.97	513353.60	0.001	15.20
Total	487039.94			564556960.14	1	16719.79

From the table above, the total Fx is obtained 16719.79 kN, which means that it is the same with the value of V and fulfills the requirement.

12. Horizontal distribution of seismic forces (Vx)

The design seismic level shear at all levels (Vx, in kN) is determined from Equation 3.22. The calculation examples are as follows.

$$V_{15} = \sum_{15}^{15} F_{15} = F_{15} = 2393.82 \text{ kN}$$

$$V_{14} = \sum_{14}^{15} F_{14} = V_{15} + F_{14} = 2393.82 + 2743.77 = 5137.59 \text{ kN}$$

$$V_{13} = \sum_{13}^{15} F_{13} = V_{14} + F_{13} = 5137.59 + 2370.59 = 7508.18 \text{ kN}$$

This calculation process continues until the first story. Then, the value of Vx on the first story must be equal to the total value of Fx. The final values can be seen in the following table.

Table 5.14 Vx Values of Each Story

Story	Wx (kN)	hx (m)	k	$W_xh_x^k$ (kNm)	Cvx	Fx (kN)	Vx (kN)
15	25104.91	60.00	1.97	80829363.33	0.14	2393.82	2393.82
14	32970.33	56.00	1.97	92645520.24	0.16	2743.77	5137.59
13	32970.33	52.00	1.97	80044734.18	0.14	2370.59	7508.18
12	32970.33	48.00	1.97	68352838.17	0.12	2024.32	9532.50
11	32970.33	44.00	1.97	57571820.54	0.10	1705.03	11237.53
10	32970.33	40.00	1.97	47703848.08	0.08	1412.79	12650.32
9	32970.33	36.00	1.97	38751300.81	0.07	1147.65	13797.97
8	32970.33	32.00	1.97	30716817.82	0.05	909.70	14707.67
7	32970.33	28.00	1.97	23603359.84	0.04	699.03	15406.70
6	32970.33	24.00	1.97	17414297.33	0.03	515.74	15922.44
5	32970.33	20.00	1.97	12153540.61	0.02	359.94	16282.38
4	32970.33	16.00	1.97	7825743.78	0.01	231.77	16514.14
3	32970.33	12.00	1.97	4436651.93	0.01	131.39	16645.54
2	32970.33	8.00	1.97	1993769.87	0.004	59.05	16704.58
1	33320.80	4.00	1.97	513353.60	0.001	15.20	16719.79
Total	487039.94			564556960.14	1	16719.79	

From the table above, it is found that the total accumulated value of Vx in the first story is the same as the total value of Fx. This means that the values have fulfilled the requirement, and the initial seismic design is a success.

5.3.2 Response Spectrum

The requirements for response spectrum acceleration design (Sa) by SNI 1726:2019 are shown by Equations 3.23 until 3.26. The calculations are as follows.

1. For $T < T_0$

   Taking T = 0 s, the calculation of Sa is as follows.

   $$\text{Sa} = \text{SDS} \times \left(0.4 + 0.6\frac{\text{T}}{\text{T}_0}\right) = 0.780 \times \left(0.4 + 0.6\frac{0}{0.155}\right) = 0.312\ \text{s}$$

2. For $T \geq T_0$ and $T \leq T_S$

   $$\text{Sa} = \text{SDS} = 0.780\ \text{s}$$

3. For $T_S \leq T \leq T_L$

   Taking T = 1 s, the calculation of Sa is as follows.

   $$\text{Sa} = \frac{\text{SD1}}{\text{T}} = \frac{0.606}{1} = 0.606\ \text{s}$$

   The calculation continues until T = 6 s.

4. For $T \geq T_L$

   Taking T = 7 s, the calculation of Sa is as follows.

   $$\text{Sa} = \frac{\text{SD1}\times\text{T}_\text{L}}{\text{T}^2} = \frac{0.606\times6}{7^2} = 0.074\ \text{s}$$

   The results can be seen in the following table.

Table 5.15 Response Spectrum Acceleration Design (Sa) Values

T (s)	Sa (G)
0	0.312
0.155	0.780
0.777	0.780
1	0.606
2	0.303
3	0.202
4	0.152
5	0.121
6	0.101

From results of the table above, the following graph of elastic response spectrum is obtained.

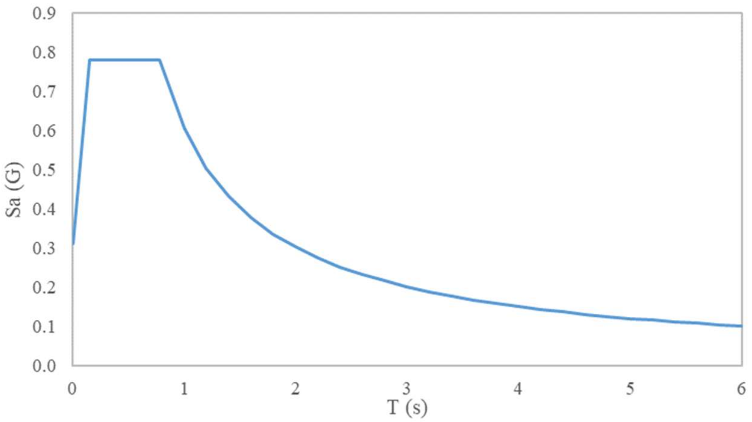


Figure 5.2 Response Spectrum Graph

(Source: Ms. Excel Analysis)

Based on the figure above, it is concluded that the elastic response spectrum already fulfills the requirements by SNI 1726:2019.

5.4 Stage 1 Building Performance under Earthquake Lateral Force (ELF) with Fixed Support

5.4.1 Irregularity Analysis

As explained in Subchapter 3.7.1, the vertical irregularity analysis is not carried out in this study because the vertical configuration of the building is already considered regular without any height differences. Meanwhile, the horizontal irregularity analysis of the building is carried out by using the data taken from the ETABS model with the load combination of 1D + 0.5L + 1EX for X direction and 1D + 0.5L + 1EY for Y direction, in which the structure is modelled using fixed support at the base. These data are δA (horizontal displacement of point A) and δB (horizontal displacement of point B), which can be seen in the following table. ΔA and ΔB show the difference between the horizontal displacement of a point in one story and the story underneath which are under consideration. Δavg shows the average difference value between ΔA and ΔB, while Δmax shows the maximum difference value between the two. To check whether the horizontal configuration of a story is considered regular, the maximum difference value (Δmax) is divided by the average difference value (Δavg). The result of the analysis in X direction can be seen in the following table.

Table 5.16 Horizontal Irregularity Analysis of X Direction

Story	δA	δB	ΔA	ΔB	Δavg	Δmax	Check	
	(mm)	(mm)					Δmax/Δavg	Status
15	102.602	102.603	2.439	2.437	2.438	2.439	1.00	Regular
14	100.163	100.166	3.418	3.417	3.418	3.418	1.00	Regular
13	96.745	96.749	4.550	4.548	4.549	4.550	1.00	Regular
12	92.195	92.201	5.642	5.640	5.641	5.642	1.00	Regular
11	86.553	86.561	6.616	6.616	6.616	6.616	1.00	Regular
10	79.937	79.945	7.449	7.447	7.448	7.449	1.00	Regular
9	72.488	72.498	8.132	8.132	8.132	8.132	1.00	Regular
8	64.356	64.366	8.673	8.672	8.673	8.673	1.00	Regular
7	55.683	55.694	9.071	9.070	9.071	9.071	1.00	Regular
6	46.612	46.624	9.323	9.324	9.324	9.324	1.00	Regular
5	37.289	37.300	9.398	9.398	9.398	9.398	1.00	Regular
4	27.891	27.902	9.211	9.212	9.212	9.212	1.00	Regular
3	18.680	18.690	8.549	8.553	8.551	8.553	1.00	Regular
2	10.131	10.137	6.923	6.926	6.925	6.926	1.00	Regular
1	3.208	3.211	3.208	3.211	3.210	3.211	1.00	Regular

From the table above, it is found that the building plan in X direction is considered regular. Meanwhile, the result of the analysis in Y direction can be seen in the following table.

Table 5.17 Horizontal Irregularity Analysis of Y Direction

Story	δA	δB	ΔA	ΔB	Δavg	Δmax	Check	
	(mm)	(mm)					Δmax/Δavg	Status
15	101.075	101.075	2.343	2.343	2.343	2.343	1.00	Regular
14	98.732	98.732	3.376	3.376	3.376	3.376	1.00	Regular
13	95.356	95.356	4.515	4.515	4.515	4.515	1.00	Regular
12	90.841	90.841	5.577	5.577	5.577	5.577	1.00	Regular
11	85.264	85.264	6.502	6.502	6.502	6.502	1.00	Regular
10	78.762	78.762	7.274	7.274	7.274	7.274	1.00	Regular
9	71.488	71.488	7.902	7.902	7.902	7.902	1.00	Regular
8	63.586	63.586	8.391	8.391	8.391	8.391	1.00	Regular
7	55.195	55.195	8.754	8.754	8.754	8.754	1.00	Regular
6	46.441	46.441	8.994	8.993	8.994	8.994	1.00	Regular
5	37.447	37.448	9.100	9.100	9.100	9.100	1.00	Regular
4	28.347	28.348	9.015	9.016	9.016	9.016	1.00	Regular
3	19.332	19.332	8.561	8.561	8.561	8.561	1.00	Regular
2	10.771	10.771	7.221	7.221	7.221	7.221	1.00	Regular
1	3.550	3.550	3.550	3.550	3.550	3.550	1.00	Regular

From the table above, it is found that the building plan in Y direction is considered regular. Both X and Y direction analyses show that no horizontal irregularity is found in the building configuration.

5.4.2 Story Drift

The analysis of story drift uses the data taken from the story drift of the building in the center of mass with the load combination of 1D + 0.5L + 1EX for X direction and 1D + 0.5L + 1EY for Y direction. Then, the drift between one story and the story underneath is calculated using Equation 3.28. The example of calculation for the 15th story with the load combination of 1D + 0.5L + 1EX for X direction is as follows.

$$\Delta_{15} \quad = \frac{(\delta_{15} - \delta_{14})Cd}{I_E}$$

$$= \frac{(102.602 - 100.164)5.5}{1}$$

$$= 13.41 \text{ mm}$$

Meanwhile, the allowable story drift is calculated based on Table 3.14. Because the building is categorized in the II category, and the structure is included as “All the other structures”, the equation used to calculate the allowable story drift is 0.020hsx. The calculation example is as follows.

$$\text{Allowable } \Delta \quad = 0.020 \times h_{sx}$$

$$= 0.020 \times 4000$$

$$= 80 \text{ mm}$$

After calculating the story drift of each story and the allowable story drift, the analysis results are obtained in both X and Y directions. The story drift obtained must not surpass the allowable story drift value to be accepted. The result of the analysis in X direction with load combination 1D + 0.5L + 1EX can be seen in the following table.

Table 5.18 Stage 1 Story Drift Analysis of X Direction with Load Combination 1D + 0.5L + 1EX

Story	hsx	δ (Ux)	Δ	Allowable Δ	Check
	(mm)	(mm)	(mm)	(mm)	
Story15	4000	102.602	13.409	80	OK
Story14	4000	100.164	18.794	80	OK

Story13	4000	96.747	25.020	80	OK
Story12	4000	92.198	31.026	80	OK
Story11	4000	86.557	36.388	80	OK
Story10	4000	79.941	40.964	80	OK
Story9	4000	72.493	44.726	80	OK
Story8	4000	64.361	47.696	80	OK
Story7	4000	55.689	49.891	80	OK
Story6	4000	46.618	51.277	80	OK
Story5	4000	37.295	51.695	80	OK
Story4	4000	27.896	50.661	80	OK
Story3	4000	18.685	47.031	80	OK
Story2	4000	10.134	38.088	80	OK
Story1	4000	3.209	17.650	80	OK

From the table above, it is found that the story drift of X direction with earthquake load in X direction does not surpass the value of allowable drift of 80 mm. Meanwhile, the result of the analysis in Y direction with load combination 1D + 0.5L + 1EY can be seen in the following table.

Table 5.19 Stage 1 Story Drift Analysis of Y Direction with Load Combination 1D + 0.5L + 1EY

Story	hsx	δ (Uy)	Δ	Allowable Δ	Check
	(mm)	(mm)	(mm)	(mm)	
Story15	4000	101.075	12.887	80	OK
Story14	4000	98.732	18.568	80	OK
Story13	4000	95.356	24.833	80	OK
Story12	4000	90.841	30.674	80	OK
Story11	4000	85.264	35.761	80	OK
Story10	4000	78.762	40.007	80	OK
Story9	4000	71.488	43.461	80	OK
Story8	4000	63.586	46.151	80	OK
Story7	4000	55.195	48.147	80	OK
Story6	4000	46.441	49.467	80	OK
Story5	4000	37.447	50.045	80	OK
Story4	4000	28.348	49.588	80	OK
Story3	4000	19.332	47.086	80	OK
Story2	4000	10.771	39.716	80	OK
Story1	4000	3.550	19.525	80	OK

From the table above, it is found that the story drift of Y direction with earthquake load in Y direction does not surpass the value of allowable drift of 80 mm. Finally, after checking the story drift values compared to the allowable story drift in both directions, the building can be determined as safe according to the SNI 1726:2019 story drift criteria.

To visualize the results, the story drift values are depicted into a graph for each direction.

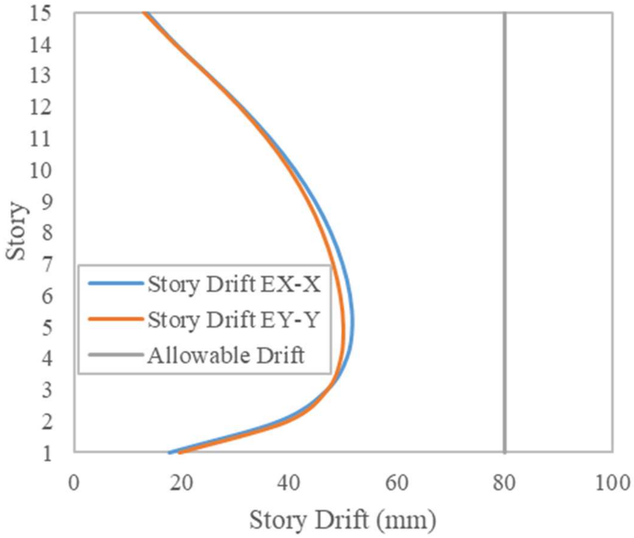


Figure 5.3 Stage 1 Story Drift Comparison in X and Y Directions

From the figure, it can be concluded that the story drift in X and Y directions are not extremely distinctive, and that both have fulfilled the requirements of the allowable drift.

5.4.3 P-Delta Effect

The stability coefficient (θ) is calculated and must not exceed the maximum coefficient value to determine whether the P-delta effect is safe. To calculate the stability coefficient, the Px values (total vertical design load at and above the x-level) of each story of the building must be obtained from the ETABS model. The following table shows the value of Px in each story of the building.

Table 5.20 Px Values of Each Story

Story	Px (kN)
15	26591.53
14	64339.57
13	102087.60
12	139835.64
11	177583.68

10	215331.72
9	253079.75
8	290827.79
7	328575.83
6	366323.87
5	404071.90
4	441819.94
3	479567.98
2	517316.02
1	555657.53

Meanwhile, the Vx values (a seismic shear force acting at and above the x-level) must also be determined to calculate the stability coefficient of each story. To calculate Vx, the Fx value of each story is added with the Vx value above the story at x-level. The calculation example is as follows.

$V_{15} = F_{15} = 2393.82 \text{ kN}$

$V_{14} = V_{15} + F_{14} = 2393.82 + 2743.77 = 5137.59 \text{ kN}$

The calculation of Vx continues until the first story. The results can be seen in the following table.

Table 5.21 Vx Values of Each Story

Story	Fx (kN)	Vx (kN)
15	2393.82	2393.82
14	2743.77	5137.59
13	2370.59	7508.18
12	2024.32	9532.50
11	1705.03	11237.53
10	1412.79	12650.32
9	1147.65	13797.97
8	909.70	14707.67
7	699.03	15406.70
6	515.74	15922.44
5	359.94	16282.38
4	231.77	16514.14
3	131.39	16645.54
2	59.05	16704.58
1	15.20	16719.79
Total	16719.79	

From the table above, it is found that the total accumulated value of Vx in the first story is the same as the total value of Fx, which means it fulfills the requirement. After obtaining the values of Px and Vx, the stability coefficient (θ) can be determined using Equation 3.30. The calculation example in X direction with load combination 1D + 0.5L + 1EX is as follows.

$$\theta_{15} = \frac{P_{15}\cdot\Delta_{15}\cdot I_e}{V_{15}\cdot h_{sx}\cdot Cd} = \frac{26591.53\cdot 13.41\cdot 1}{2393.82\cdot 4000\cdot 5.5} = 0.00677$$

$$\theta_{14} = \frac{P_{14}\cdot\Delta_{14}\cdot I_e}{V_{14}\cdot h_{sx}\cdot Cd} = \frac{64339.57\cdot 18.79\cdot 1}{5137.59\cdot 4000\cdot 5.5} = 0.01070$$

The stability coefficient (θ) must not exceed θ_{max} which is determined using Equation 3.31. The calculation is as follows.

$$\theta_{max} = \frac{0.5}{\beta\cdot Cd} \leq 0.25$$

$$\theta_{max} = \frac{0.5}{1\cdot 5.5} \leq 0.25$$

$$\theta_{max} = 0.09091 < 0.25$$

The stability coefficient calculation continues until the first story. The coefficient of each story must not exceed the maximum coefficient value. The result of the analysis in X direction with load combination 1D + 0.5L + 1EX can be seen in the following table.

Table 5.22 Stage 1 P-Delta Analysis of Each Story in X Direction with Load Combination 1D + 0.5L + 1EX

Story	hsx	Δ	Px	Vx	θx	θmax	Check
	(mm)	(mm)	(kN)	(kN)			
15	4000	13.41	26591.53	2393.82	0.00677	0.09091	OK
14	4000	18.79	64339.57	5137.59	0.01070	0.09091	OK
13	4000	25.02	102087.60	7508.18	0.01546	0.09091	OK
12	4000	31.03	139835.64	9532.50	0.02069	0.09091	OK
11	4000	36.39	177583.68	11237.53	0.02614	0.09091	OK
10	4000	40.96	215331.72	12650.32	0.03169	0.09091	OK
9	4000	44.73	253079.75	13797.97	0.03729	0.09091	OK
8	4000	47.70	290827.79	14707.67	0.04287	0.09091	OK
7	4000	49.89	328575.83	15406.70	0.04836	0.09091	OK
6	4000	51.28	366323.87	15922.44	0.05362	0.09091	OK
5	4000	51.69	404071.90	16282.38	0.05831	0.09091	OK
4	4000	50.66	441819.94	16514.14	0.06161	0.09091	OK
3	4000	47.03	479567.98	16645.54	0.06159	0.09091	OK
2	4000	38.09	517316.02	16704.58	0.05361	0.09091	OK
1	4000	17.65	555657.53	16719.79	0.02666	0.09091	OK

From the table above, it is found that the stability coefficients of each story in the X direction with earthquake load in X direction do not surpass the maximum value of 0.09091. Meanwhile, the result of the analysis in Y direction with load combination 1D + 0.5L + 1EY can be seen in the following table.

Table 5.23 Stage 1 P-Delta Analysis of Each Story in Y Direction with Load Combination 1D + 0.5L + 1EY

Story	hsx	Δ	Px	Vx	θx	θmax	Check
	(mm)	(mm)	(kN)	(kN)			
15	4000	12.89	26591.53	2393.82	0.00651	0.09091	OK
14	4000	18.57	64339.57	5137.59	0.01057	0.09091	OK
13	4000	24.83	102087.60	7508.18	0.01535	0.09091	OK
12	4000	30.67	139835.64	9532.50	0.02045	0.09091	OK
11	4000	35.76	177583.68	11237.53	0.02569	0.09091	OK
10	4000	40.01	215331.72	12650.32	0.03095	0.09091	OK
9	4000	43.46	253079.75	13797.97	0.03623	0.09091	OK
8	4000	46.15	290827.79	14707.67	0.04148	0.09091	OK
7	4000	48.15	328575.83	15406.70	0.04667	0.09091	OK
6	4000	49.47	366323.87	15922.44	0.05173	0.09091	OK
5	4000	50.04	404071.90	16282.38	0.05645	0.09091	OK
4	4000	49.59	441819.94	16514.14	0.06030	0.09091	OK
3	4000	47.09	479567.98	16645.54	0.06166	0.09091	OK
2	4000	39.72	517316.02	16704.58	0.05591	0.09091	OK
1	4000	19.53	555657.53	16719.79	0.02949	0.09091	OK

From the table above, it is found that the stability coefficients of each story in the Y direction with earthquake load in Y direction do not surpass the maximum value of 0.09091. Finally, after checking the stability coefficient values compared to the maximum coefficient in both directions, the building can be determined as safe according to the SNI 1726:2019 P-delta effect criteria.

To visualize the results, the stability coefficient values are depicted into a graph for both directions.

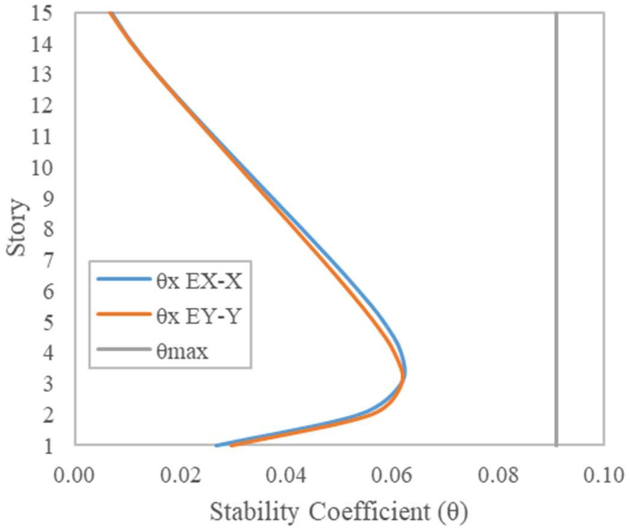


Figure 5.4 Stage 1 P-Delta Effect Comparison in EX-X and EY-Y Directions

From the figure, it can be concluded that the stability coefficient in the analysis of P-delta effect in X and Y directions are not extremely distinctive, and that both have fulfilled the requirements of the maximum coefficient value.

5.5 Stage 2 Computation of Internal Forces for Member Design with Fixed Support

5.5.1 Redundancy Factor

A redundancy factor (ρ) shall be assigned to the structure above the isolation system based on the requirements. As mentioned in Subchapter 3.8.1, the redundancy factor (ρ) value used in this stage of analysis may be considered as equal to 1.0. This is because albeit the structure's seismic design category is classified as D, the building is isolated, does not have torsional horizontal irregularities, and the structural plan has more than two spans of beams in each direction, which fulfills the requirements of SNI 1726:2019. However, the redundancy factor used in this design is taken as 1.3, for if loosely analyzed, the structure's seismic design category is still classified as D. Moreover, the main concern of the study is to compare the foundation instead of focusing on the design of the building or superstructure. Hence, the redundancy factor used in the analysis equals to 1.3.

5.5.2 Torsion Analysis

Torsion analysis includes natural torsion and torsion amplification or accidental torsion. These torsions cause natural and accidental eccentricity as well. The eccentricities used in the load combinations are a combination of eccentricities due to natural and accidental torsions.

According to Equations 3.34 and 3.35, the value of eccentricity is determined by adding eo (eccentricity due to natural torsion) value with et (additional eccentricity due to accidental torsion), in either direction. The eo values obtained from the ETABS model can be seen in the following table.

Table 5.24 Values of Eccentricity due to Natural Torsion

Story	eox (m)	eoy (m)
15	35.9997	14.9977
14	36	15
13	36	15
12	36	15
11	36	15
10	36	15
9	36	15
8	36	15
7	36	15
6	36	15
5	36	15
4	36	15
3	36	15
2	36	15
1	36	15.0032

Meanwhile, the additional eccentricity due to accidental torsion values can be determined using Equations 3.32 and 3.33. The calculation example is as follows.

L building = 72 m

B building = 30 m

$e_{tx} = 5\% \cdot L_{building} = 5\% \cdot 72 = 3.6 \text{ m}$

$e_{ty} = 5\% \cdot B_{building} = 5\% \cdot 30 = 1.5 \text{ m}$

After obtaining the additional eccentricity due to accidental torsion values, the next step is to determine the final values of eccentricity by adding the additional eccentricity to the eccentricity due to natural torsion values in center of mass and rigidity using Equations 3.34 and 3.35. The calculation example for the 15th story in X direction is as follows.

$e_{x+} = e_{ox} + e_{tx} = 35.9997 + 3.6 = 39.5997$ m

$e_{x-} = e_{ox} - e_{tx} = 35.9997 - 3.6 = 32.3997$ m

Meanwhile, the calculation example for the 15th story in Y direction is as follows.

$e_{y+} = e_{oy} + e_{ty} = 14.9977 + 1.5 = 16.4977$ m

$e_{y-} = e_{oy} - e_{ty} = 14.9977 - 1.5 = 13.4977$ m

The results of the calculation of eccentricity in both directions can be seen in the following table.

Table 5.25 Additional Eccentricity Values of Each Story in X and Y Directions

Story	ex+ (m)	ex- (m)	ey+ (m)	ey- (m)
15	39.5997	32.3997	16.4977	13.4977
14	39.6	32.4	16.5	13.5
13	39.6	32.4	16.5	13.5
12	39.6	32.4	16.5	13.5
11	39.6	32.4	16.5	13.5
10	39.6	32.4	16.5	13.5
9	39.6	32.4	16.5	13.5
8	39.6	32.4	16.5	13.5
7	39.6	32.4	16.5	13.5
6	39.6	32.4	16.5	13.5
5	39.6	32.4	16.5	13.5
4	39.6	32.4	16.5	13.5
3	39.6	32.4	16.5	13.5
2	39.6	32.4	16.5	13.5
1	39.6	32.4	16.5032	13.5032

The additional eccentricity values shown in the table above are then incorporated into the load combinations in the ETABS model.

5.5.3 Loading Scheme

One of the examples of a load combination with the inserted values of SDS, redundancy factor, and additional eccentricity can be seen as follows.

Comb1 = (1.2 + 0.2SDS) D + 0.5L + ρ QEx+TT + ρ 30%QEy
= (1.2 + 0.2(0.780)) D + 0.5L + 1.3 QEx+TT + 1.3 30%QEy
= 1.36D + 0.5L + 1.3QEx+TT + 0.39QEy

Finally, with all the inserted values into the load combinations, the results of the second stage load combination are as follows.

1. 1.36D + 0.5L + 1.3QEx+TT + 0.39QEy
2. 1.36D + 0.5L + 1.3QEx+TT - 0.39QEy
3. 1.36D + 0.5L - 1.3QEx+TT + 0.39QEy
4. 1.36D + 0.5L - 1.3QEx+TT - 0.39QEy
5. 1.36D + 0.5L + 1.3QEx-TT + 0.39QEy
6. 1.36D + 0.5L + 1.3QEx-TT - 0.39QEy
7. 1.36D + 0.5L - 1.3QEx-TT + 0.39QEy
8. 1.36D + 0.5L - 1.3QEx-TT - 0.39QEy
9. 1.36D + 0.5L + 1.3QEy+TT + 0.39QEx
10. 1.36D + 0.5L + 1.3QEy+TT - 0.39QEx
11. 1.36D + 0.5L - 1.3QEy+TT + 0.39QEx
12. 1.36D + 0.5L - 1.3QEy+TT - 0.39QEx
13. 1.36D + 0.5L + 1.3QEy-TT + 0.39QEx
14. 1.36D + 0.5L + 1.3QEy-TT - 0.39QEx
15. 1.36D + 0.5L - 1.3QEy-TT + 0.39QEx
16. 1.36D + 0.5L - 1.3QEy-TT - 0.39QEx
17. 0.74D + 1.3QEx+TT + 0.39QEy
18. 0.74D + 1.3QEx+TT - 0.39QEy
19. 0.74D - 1.3QEx+TT + 0.39QEy
20. 0.74D - 1.3QEx+TT - 0.39QEy
21. 0.74D + 1.3QEx-TT + 0.39QEy
22. 0.74D + 1.3QEx-TT - 0.39QEy
23. 0.74D - 1.3QEx-TT + 0.39QEy
24. 0.74D - 1.3QEx-TT - 0.39QEy
25. 0.74D + 1.3QEy+TT + 0.39QEx
26. 0.74D + 1.3QEy+TT - 0.39QEx

27. 0.74D - 1.3QEy+TT + 0.39QEx
28. 0.74D - 1.3QEy+TT - 0.39QEx
29. 0.74D + 1.3QEy-TT + 0.39QEx
30. 0.74D + 1.3QEy-TT - 0.39QEx
31. 0.74D - 1.3QEy-TT + 0.39QEx
32. 0.74D - 1.3QEy-TT - 0.39QEx

These load combinations are then enveloped in the ETABS program according to each of the directions. Hence, the loading scheme in the second stage results in the envelope of earthquake load combinations in EX and EY directions.

5.5.4 Equivalent Lateral Force (ELF) Analysis

Equivalent static analysis is a structural analysis method with earthquake vibrations which are modeled as static horizontal loads acting on the building's mass centers. The horizontal force acting on the building's mass centers is only static in nature, meaning that the magnitude and location are fixed, while the dynamic load varies in intensity according to time (dynamic). These horizontal forces are only equivalent in characteristic as a substitute/representation of the dynamic load effect that occurs during an earthquake. Therefore, these horizontal forces are generally referred to as static equivalent horizontal forces/loads. In the ETABS model used for this analysis, the structure is modeled using fixed support at the base.

In the ELF analysis, the story drift and P-delta effect of X and Y directions are analyzed and will further be compared with the RS analysis. The load combinations used to analyze the story drift are the envelope of earthquake load combinations in both EX and EY directions. The story drift analysis is as follows.

The analysis of story drift uses the data taken from the story drift of the building in the center of mass. Then, the difference in the drift between one story and the story underneath is calculated using Equation 3.28. The example of calculation for the 15th story with the envelope of earthquake load combinations in EX direction is as follows.

$$\Delta_{15}=\frac{(\delta_{15}-\delta_{14})\mathrm{Cd}}{\mathrm{I_E}}=\frac{(133.383-1\quad .213)5.5}{1}=17.435\text{ mm}$$

Meanwhile, the allowable story drift is calculated based on Table 3.14. Because the building is categorized in the II category, and the structure is included as “All the other structures”, the equation used to calculate the allowable story drift is 0.020hsx. The calculation example is as follows.

Allowable $\Delta = 0.020 \times \mathrm{h_{sx}} = 0.020 \times 4000 = 80\ \mathrm{mm}$

After calculating the story drift of each story and the allowable story drift, the analysis results are obtained with the envelope of earthquake load combinations in both EX and EY directions. The story drift obtained must not surpass the allowable story drift to be accepted. The result of the analysis with the envelope of earthquake load combinations in EX direction can be seen in the following table.

Table 5.26 Stage 2 ELF Story Drift Analysis of X Direction with the Envelope of Earthquake Load Combinations in EX Direction

Story	hsx	δ (Ux)	Δ	Allowable Δ	Check
	(mm)	(mm)	(mm)	(mm)	
Story15	4000	133.383	17.435	80	OK
Story14	4000	130.213	24.431	80	OK
Story13	4000	125.771	32.522	80	OK
Story12	4000	119.858	40.337	80	OK
Story11	4000	112.524	47.306	80	OK
Story10	4000	103.923	53.251	80	OK
Story9	4000	94.241	58.146	80	OK
Story8	4000	83.669	62.007	80	OK
Story7	4000	72.395	64.856	80	OK
Story6	4000	60.603	66.660	80	OK
Story5	4000	48.483	67.199	80	OK
Story4	4000	36.265	65.863	80	OK
Story3	4000	24.290	61.138	80	OK
Story2	4000	13.174	49.511	80	OK
Story1	4000	4.172	22.946	80	OK

Table 5.27 Stage 2 ELF Story Drift Analysis of Y Direction with the Envelope of Earthquake Load Combinations in EX Direction

Story	hsx	δ (Uy)	Δ	Allowable Δ	Check
	(mm)	(mm)	(mm)	(mm)	
Story15	4000	39.427	5.021	80	OK
Story14	4000	38.514	7.238	80	OK
Story13	4000	37.198	9.691	80	OK
Story12	4000	35.436	11.968	80	OK
Story11	4000	33.260	13.948	80	OK
Story10	4000	30.724	15.609	80	OK

Story9	4000	27.886	16.951	80	OK
Story8	4000	24.804	18.002	80	OK
Story7	4000	21.531	18.777	80	OK
Story6	4000	18.117	19.294	80	OK
Story5	4000	14.609	19.525	80	OK
Story4	4000	11.059	19.338	80	OK
Story3	4000	7.543	18.370	80	OK
Story2	4000	4.203	15.494	80	OK
Story1	4000	1.386	7.623	80	OK

From the tables above, it is found that the story drift values in each story of both X and Y directions with the envelope of earthquake load combinations in EX direction do not surpass the value of allowable drift of 80 mm. Meanwhile, the result of the analysis the envelope of earthquake load combinations in EY direction can be seen in the following table.

Table 5.28 Stage 2 ELF Story Drift Analysis of X Direction with the Envelope of Earthquake Load Combinations in EY Direction

Story	hsx	δ (Ux)	Δ	Allowable Δ	Check
	(mm)	(mm)	(mm)	(mm)	
Story15	4000	40.015	5.231	80	OK
Story14	4000	39.064	7.331	80	OK
Story13	4000	37.731	9.757	80	OK
Story12	4000	35.957	12.100	80	OK
Story11	4000	33.757	14.190	80	OK
Story10	4000	31.177	15.978	80	OK
Story9	4000	28.272	17.441	80	OK
Story8	4000	25.101	18.601	80	OK
Story7	4000	21.719	19.459	80	OK
Story6	4000	18.181	19.998	80	OK
Story5	4000	14.545	20.158	80	OK
Story4	4000	10.880	19.762	80	OK
Story3	4000	7.287	18.343	80	OK
Story2	4000	3.952	14.850	80	OK
Story1	4000	1.252	6.886	80	OK

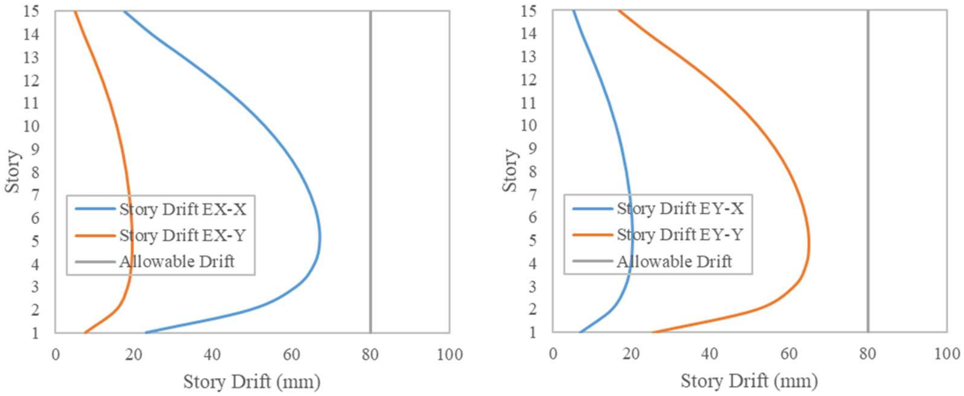
Table 5.29 Stage 2 ELF Story Drift Analysis of Y Direction with the Envelope of Earthquake Load Combinations in EY Direction

Story	hsx	δ (Uy)	Δ	Allowable Δ	Check
	(mm)	(mm)	(mm)	(mm)	
Story15	4000	131.392	16.753	80	OK
Story14	4000	128.346	24.134	80	OK
Story13	4000	123.958	32.280	80	OK

Story12	4000	118.089	39.875	80	OK
Story11	4000	110.839	46.481	80	OK
Story10	4000	102.388	52.014	80	OK
Story9	4000	92.931	56.491	80	OK
Story8	4000	82.660	60.000	80	OK
Story7	4000	71.751	62.585	80	OK
Story6	4000	60.372	64.306	80	OK
Story5	4000	48.680	65.060	80	OK
Story4	4000	36.851	64.460	80	OK
Story3	4000	25.131	61.210	80	OK
Story2	4000	14.002	51.629	80	OK
Story1	4000	4.615	25.383	80	OK

From the tables above, it is found that the story drift values in each story of both X and Y directions with the envelope of earthquake load combinations in EY direction do not surpass the value of allowable drift of 80 mm. Finally, after checking the story drift values compared to the allowable story drift in both directions, the building can be determined as safe according to the SNI 1726:2019 story drift criteria.

To visualize the results, the story drift values are depicted into a graph for each envelope of earthquake load combinations in each direction.



(a) (b)

Figure 5.5 Stage 2 ELF Story Drift Comparison in X and Y Directions with the Envelope of Earthquake Load Combinations in (a) EX Direction and (b) EY Direction

From the figure, it can be concluded that the story drift in X direction with the envelope of earthquake load combinations in EX direction is larger than in Y direction. On the other hand, the story drift in Y direction with the envelope of earthquake load combinations in EY direction is larger than in X direction. This proves that the critical direction of the building configuration is directly affected by the earthquake load direction. It can also be concluded that both earthquake load directions have fulfilled the requirements of the allowable drift.

After analyzing the story drift of ELF analysis in the second stage, the next step is to analyze the P-delta effect. The P-delta effect analysis is as follows.

In the P-delta effect analysis, the stability coefficient (θ) is calculated and must not exceed the maximum coefficient value to determine whether the P-delta effect is safe. To calculate the stability coefficient, the Px values (total vertical design load at and above the x-level) of each story of the building and Vx values (a seismic shear force acting at and above the x-level) must be obtained from the ETABS model. The stability coefficient (θ) can be determined using Equation 3.30. The calculation example of the stability coefficient in X direction with the envelope of earthquake load combinations in EX direction is as follows.

$$\theta_{15} = \frac{P_{15} \cdot \Delta_{15} \cdot I_e}{V_{15} \cdot h_{sx} \cdot Cd} = \frac{26591.53 \cdot 17.44 \cdot 1}{3111.97 \cdot 4000 \cdot 5.5} = 0.00677$$

$$\theta_{14} = \frac{P_{14} \cdot \Delta_{14} \cdot I_e}{V_{14} \cdot h_{sx} \cdot Cd} = \frac{64339.57 \cdot 24.43 \cdot 1}{6678.87 \cdot 4000 \cdot 5.5} = 0.01070$$

The stability coefficient (θ) must not exceed θ_{max} which is determined using Equation 3.31. The calculation is as follows.

$$\theta_{max} = \frac{0.5}{\beta \cdot Cd} \leq 0.25$$

$$\theta_{max} = \frac{0.5}{1 \cdot 5.5} \leq 0.25$$

$$\theta_{max} = 0.09091 < 0.25$$

The stability coefficient calculation continues until the first story. The coefficient of each story must not exceed the maximum coefficient value. The result of P-delta analysis in X direction with the envelope of earthquake load combinations in EX direction can be seen in the following table.

Table 5.30 Stage 2 ELF P-Delta Analysis of Each Story in X Direction with the Envelope of Earthquake Load Combinations in EX Direction

Story	hsx	Δ	Px	Vx	θx	θmax	Check
	(mm)	(mm)	(kN)	(kN)			
15	4000	17.44	26591.53	3111.97	0.00677	0.09091	OK
14	4000	24.43	64339.57	6678.87	0.01070	0.09091	OK
13	4000	32.52	102087.60	9760.63	0.01546	0.09091	OK
12	4000	40.34	139835.64	12392.25	0.02069	0.09091	OK
11	4000	47.31	177583.68	14608.79	0.02614	0.09091	OK
10	4000	53.25	215331.72	16445.42	0.03169	0.09091	OK
9	4000	58.15	253079.75	17937.36	0.03729	0.09091	OK
8	4000	62.01	290827.79	19119.97	0.04287	0.09091	OK
7	4000	64.86	328575.83	20028.71	0.04836	0.09091	OK
6	4000	66.66	366323.87	20699.17	0.05362	0.09091	OK
5	4000	67.20	404071.90	21167.09	0.05831	0.09091	OK
4	4000	65.86	441819.94	21468.40	0.06161	0.09091	OK
3	4000	61.14	479567.98	21639.20	0.06159	0.09091	OK
2	4000	49.51	517316.02	21715.97	0.05361	0.09091	OK
1	4000	22.95	555657.53	21735.73	0.02666	0.09091	OK

From the table above, it is found that the stability coefficient values in each story of X direction with the envelope of earthquake load combinations in EX direction do not surpass the maximum coefficient value of 0.09091. Meanwhile, the result of P-delta analysis in Y direction with the envelope of earthquake load combinations in EY direction can be seen in the following table.

Table 5.31 Stage 2 ELF P-Delta Analysis of Each Story in Y Direction with the Envelope of Earthquake Load Combinations in EY Direction

Story	hsx	Δ	Px	Vx	θx	θmax	Check
	(mm)	(mm)	(kN)	(kN)			
15	4000	16.75	26591.53	3111.97	0.00651	0.09091	OK
14	4000	24.13	64339.57	6678.87	0.01057	0.09091	OK
13	4000	32.28	102087.60	9760.63	0.01535	0.09091	OK
12	4000	39.88	139835.64	12392.25	0.02045	0.09091	OK
11	4000	46.48	177583.68	14608.79	0.02568	0.09091	OK
10	4000	52.01	215331.72	16445.42	0.03096	0.09091	OK
9	4000	56.49	253079.75	17937.36	0.03623	0.09091	OK
8	4000	60.00	290827.79	19119.97	0.04148	0.09091	OK
7	4000	62.58	328575.83	20028.71	0.04667	0.09091	OK
6	4000	64.31	366323.87	20699.17	0.05173	0.09091	OK
5	4000	65.06	404071.90	21167.09	0.05645	0.09091	OK
4	4000	64.46	441819.94	21468.40	0.06030	0.09091	OK
3	4000	61.21	479567.98	21639.20	0.06166	0.09091	OK

2	4000	51.63	517316.02	21715.97	0.05590	0.09091	OK
1	4000	25.38	555657.53	21735.73	0.02949	0.09091	OK

From the table above, it is found that the stability coefficient values in each story of Y direction with the envelope of earthquake load combinations in EY direction do not surpass the maximum coefficient value of 0.09091. Finally, after checking the stability coefficient values compared to the maximum coefficient in both directions, the building can be determined as safe according to the SNI 1726:2019 P-delta effect criteria.

To visualize the results, the stability coefficient values are depicted into a graph for both directions.

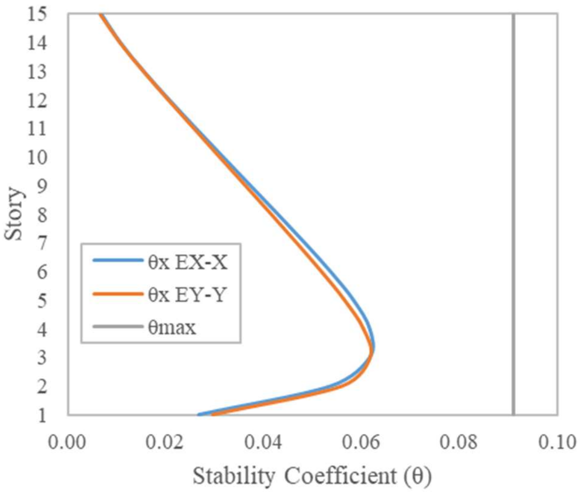


Figure 5.6 Stage 2 ELF P-Delta Effect Comparison in EX-X and EY-Y Directions

From the figure, it can be concluded that the stability coefficient in the analysis of P-delta effect in X and Y directions are not extremely distinctive, and that both have fulfilled the requirements of the maximum coefficient value.

5.5.5 Base Shear Scaling

SNI 1726:2019 requires that if the fundamental period of the analysis results is greater than CuTa in a certain direction, then the period of the T structure must be taken as equal to CuTa.

In Subchapter 5.3.1, the approach period (Ta), upper bound of the calculated period (CuTa), and fundamental period obtained from the ETABS model (Tc) have been determined. The values are as follows.

Ta = 1.86 s

CuTa = 2.60 s

Tc = 2.45 s

Because Ta < Tc < CuTa, hence the fundamental period used for the structural analysis is equal to the Tc, which is 2.45 s. Meanwhile, with the values of R, Ω, and Cd obtained in Subchapter 5.3.1, the I/R value that will be used to analyze base shear scaling can be determined.

Importance factor, Ie = 1

Response modification coefficient, R = 8

System exceeding strength factor, Ω = 3

Deflection magnification factor, Cd = 5.5

$$\frac{\mathrm{I}}{\mathrm{R}}\text{(in G)} = \frac{1}{8} = 0.125\mathrm{G}$$

$$= 0.125 \times 9.81$$

$$= 1.226\,\frac{\mathrm{m}}{\mathrm{s}^2} = 1225.831\ \mathrm{mm/s^2}$$

In the ETABS model itself, this value of 1225.831 mm/s^2 is inserted into the load case scale factor of response spectrum in both X and Y directions.

If the combined response for the base shear force resulting from the analysis of variance (Vt) is less than 100% of the shear force (V) calculated using the equivalent static method, then the force must be multiplied by V/Vt, where V is the calculated equivalent static base shear according to SNI 1726:2019, and Vt is the base shear force obtained from the analysis of the combination of variances. The known data to analyze base shear scaling are as follows.

Cs = 0.034G

W = 487039.94 kN

V = 16719.79 kN

Meanwhile, the Vt values in X and Y directions obtained from the ETABS model are as follows.

Vt x = 12445.10 kN

Vt y = 12591.01 kN

The scale factors can be determined by dividing the V value with the Vt values of each direction. The calculation is as follows.

SF x = V/Vt x

= 16719.79/12445.10 = 1.343

SF y = V/Vt y

= 16719.79/12591.01 = 1.328

If the scale factors (SF) are multiplied by the value of I/R, the results are as follows.

$$\mathrm{SF\ x} \cdot \frac{\mathrm{I}}{\mathrm{R}} = 1.343 \cdot 1.226$$

$$= 1.647\ \mathrm{m/s^2}$$

$$\mathrm{SF\ y} \cdot \frac{\mathrm{I}}{\mathrm{R}} = 1.328 \cdot 1.226$$

$$= 1.628\ \mathrm{m/s^2}$$

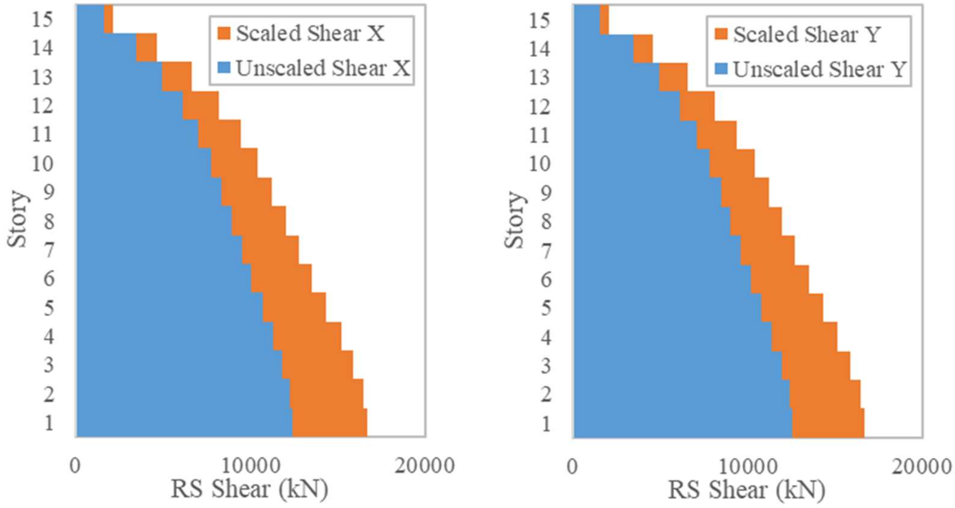
With the scale factors (SF) of each direction obtained, the next step is to export the unscaled shear forces from the ETABS model structural analysis with response spectrum load cases, which then will be scaled by multiplying the shear forces with the scale factors (SF) of each direction. The results of unscaled and scaled shear forces of each story can be seen in the following table.

Table 5.32 Scaled Story Shear Results

Story	X Direction, SF = 1.343		Y Direction, SF = 1.328	
	Unscaled Shear	Scaled Shear	Unscaled Shear	Scaled Shear
15	1608.00	2160.32	1575.32	2091.88
14	3496.00	4696.82	3458.53	4592.63
13	4963.67	6668.61	4956.57	6581.90
12	6134.23	8241.24	6152.48	8169.97
11	7049.57	9470.98	7092.46	9418.18
10	7780.24	10452.62	7843.55	10415.57
9	8392.76	11275.53	8468.24	11245.10
8	8947.41	12020.70	9034.16	11996.60
7	9504.27	12768.83	9592.81	12738.43
6	10085.08	13549.14	10180.06	13518.25
5	10698.98	14373.91	10794.90	14334.70
4	11309.46	15194.08	11414.69	15157.74
3	11854.13	15925.83	11971.47	15897.09

2	12268.95	16483.13	12397.65	16463.02
1	12445.10	16719.79	12591.01	16719.79

Finally, to visualize the results, the results are depicted into a graph as a comparison. The graphs/diagrams can be seen as follows.



(a) (b)

Figure 5.7 Unscaled and Scaled Story Shear Comparison in (a) X Direction and (b) Y Direction

From the figure, it can be concluded that the story shear forces of X and Y directions are not noticeably distinctive, both in the unscaled and scaled values.

5.5.6 Response Spectrum (RS) Analysis

In the RS analysis, the story drift and P-delta effect of X and Y directions are analyzed and will further be compared with the ELF analysis. In the ETABS model used in this analysis, the structure is modeled using fixed support at the base. The story drift analysis is as follows.

The analysis of story drift uses the data taken from the drift of the building in the center of mass with the load cases of response spectrum (RS) in X and Y directions. The response spectrum modal combination method used is Complete Squares Combination (CQC). Then, the difference in the drift between one story and the story underneath is calculated using Equation 3.28. The example of calculation for the 15th story with the load case of RSX is as follows.

$$\Delta_{15} = \frac{(\delta_{15}-\delta_{14})\mathrm{Cd}}{\mathrm{I_E}}$$

$$= \frac{(60.572 \qquad .385)5.5}{1}$$

$$= 6.364\ \mathrm{mm}$$

Meanwhile, the allowable story drift is calculated based on Table 3.14. Because the building is categorized in the II category, and the structure is included as "All the other structures", the equation used to calculate the allowable story drift is 0.020hsx. The calculation example is as follows.

$$\text{Allowable } \Delta = 0.020 \times \mathrm{h_{sx}}$$

$$= 0.020 \times 4000$$

$$= 80\ \mathrm{mm}$$

After calculating the story drift of each story and the allowable story drift, the analysis results are obtained in both X and Y directions. The story drift obtained must also be of a smaller value than the allowable story drift to be accepted. The result of the analysis in X direction with load case RSX can be seen in the following table.

Table 5.33 Stage 2 RS Story Drift Analysis of X Direction with Load Case RSX

Story	hsx	δ (Ux)	Δ	Allowable Δ	Check
	(mm)	(mm)	(mm)	(mm)	
Story15	4000	60.542	6.364	80	OK
Story14	4000	59.385	9.047	80	OK
Story13	4000	57.740	12.276	80	OK
Story12	4000	55.508	15.516	80	OK
Story11	4000	52.687	18.563	80	OK
Story10	4000	49.312	21.401	80	OK
Story9	4000	45.421	24.068	80	OK
Story8	4000	41.045	26.604	80	OK
Story7	4000	36.208	29.013	80	OK
Story6	4000	30.933	31.196	80	OK
Story5	4000	25.261	32.940	80	OK
Story4	4000	19.272	33.715	80	OK
Story3	4000	13.142	32.478	80	OK
Story2	4000	7.237	27.049	80	OK
Story1	4000	2.319	12.755	80	OK

From the table above, it is found that the story drift values in the X direction with load case RSX does not surpass the value of allowable drift of 80 mm. Meanwhile, the result of the analysis in Y direction with load case RSY can be seen in the following table.

Table 5.34 Stage 2 RS Story Drift Analysis of Y Direction with Load Case RSY

Story	hsx	δ (Uy)	Δ	Allowable Δ	Check
	(mm)	(mm)	(mm)	(mm)	
Story15	4000	60.306	6.193	80	OK
Story14	4000	59.180	9.025	80	OK
Story13	4000	57.539	12.293	80	OK
Story12	4000	55.304	15.450	80	OK
Story11	4000	52.495	18.359	80	OK
Story10	4000	49.157	21.016	80	OK
Story9	4000	45.336	23.491	80	OK
Story8	4000	41.065	25.850	80	OK
Story7	4000	36.365	28.133	80	OK
Story6	4000	31.250	30.283	80	OK
Story5	4000	25.744	32.170	80	OK
Story4	4000	19.895	33.385	80	OK
Story3	4000	13.825	33.000	80	OK
Story2	4000	7.825	28.683	80	OK
Story1	4000	2.610	14.355	80	OK

From the table above, it is found that the story drift values in the Y direction with load case RSY does not surpass the value of allowable drift of 80 mm. Finally, after checking the story drift values compared to the allowable story drift in both directions, the building can be determined as safe according to the SNI 1726:2019 story drift criteria.

To visualize the results, the story drift values are depicted into a graph for each direction.

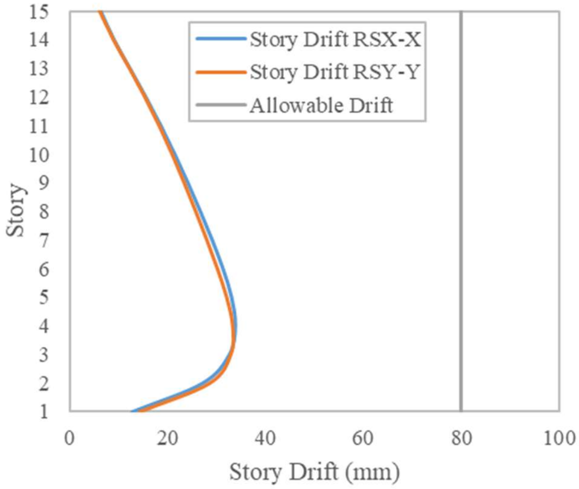


Figure 5.8 Stage 2 RS Story Drift Comparison in X and Y Directions

From the figure, it can be concluded that the story drift in X and Y directions are not extremely distinctive, and that both have fulfilled the requirements of the allowable drift.

After analyzing the story drift of RS analysis in the second stage, the next step is to analyze the P-delta effect. The P-delta effect analysis is as follows.

In the P-delta effect analysis, the stability coefficient (θ) is calculated and must not exceed the maximum coefficient value to determine whether the P-delta effect is safe. To calculate the stability coefficient, the Px values (total vertical design load at and above the x-level) of each story of the building and Vx values (a seismic shear force acting at and above the x-level) must be obtained from the ETABS model. The stability coefficient (θ) can be determined using Equation 3.30. The calculation example in X direction with load case RSX is as follows.

$$\theta_{15} = \frac{P_{15} \cdot \Delta_{15} \cdot I_e}{V_{15} \cdot h_{sx} \cdot Cd} = \frac{26591.53 \cdot 6.36 \cdot 1}{1608 \cdot 4000 \cdot 5.5} = 0.00478$$

$$\theta_{14} = \frac{P_{14} \cdot \Delta_{14} \cdot I_e}{V_{14} \cdot h_{sx} \cdot Cd} = \frac{64339.57 \cdot 9.05 \cdot 1}{3496 \cdot 4000 \cdot 5.5} = 0.00757$$

The stability coefficient (θ) must not exceed θ_{max} which is determined using Equation 3.31. The calculation is as follows.

$$\theta_{max} = \frac{0.5}{\beta \cdot Cd} \leq 0.25$$

$$\theta_{max} = \frac{0.5}{1 \cdot 5.5} \leq 0.25$$

$\theta_{\max} = 0.09091 < 0.25$

The stability coefficient calculation continues until the first story. The coefficient of each story must not exceed the maximum coefficient value. The result of the analysis in X direction with load case RSX can be seen in the following table.

Table 5.35 Stage 2 RS P-Delta Analysis of Each Story in X Direction with Load Case RSX

Story	hsx	Δ	Px	Vx	θx	θmax	Check
	(mm)	(mm)	(kN)	(kN)			
15	4000	6.36	26591.53	1608.00	0.00478	0.09091	OK
14	4000	9.05	64339.57	3496.00	0.00757	0.09091	OK
13	4000	12.28	102087.60	4963.67	0.01148	0.09091	OK
12	4000	15.52	139835.64	6134.23	0.01608	0.09091	OK
11	4000	18.56	177583.68	7049.57	0.02125	0.09091	OK
10	4000	21.40	215331.72	7780.24	0.02692	0.09091	OK
9	4000	24.07	253079.75	8392.76	0.03299	0.09091	OK
8	4000	26.60	290827.79	8947.41	0.03931	0.09091	OK
7	4000	29.01	328575.83	9504.27	0.04559	0.09091	OK
6	4000	31.20	366323.87	10085.08	0.05151	0.09091	OK
5	4000	32.94	404071.90	10698.98	0.05655	0.09091	OK
4	4000	33.72	441819.94	11309.46	0.05987	0.09091	OK
3	4000	32.48	479567.98	11854.13	0.05972	0.09091	OK
2	4000	27.05	517316.02	12268.95	0.05184	0.09091	OK
1	4000	12.75	555657.53	12445.10	0.02589	0.09091	OK

From the table above, it is found that the stability coefficients of each story in the X direction with load case RSX do not surpass the maximum value of 0.09091. Meanwhile, the result of the analysis in Y direction with load case RSY can be seen in the following table.

Table 5.36 Stage 2 RS P-Delta Analysis of Each Story in Y Direction with Load Case RSY

Story	hsx	Δ	Px	Vx	θx	θmax	Check
	(mm)	(mm)	(kN)	(kN)			
15	4000	6.19	26591.53	1575.32	0.00475	0.09091	OK
14	4000	9.03	64339.57	3458.53	0.00763	0.09091	OK
13	4000	12.29	102087.60	4956.57	0.01151	0.09091	OK
12	4000	15.45	139835.64	6152.48	0.01596	0.09091	OK
11	4000	18.36	177583.68	7092.46	0.02089	0.09091	OK
10	4000	21.02	215331.72	7843.55	0.02622	0.09091	OK
9	4000	23.49	253079.75	8468.24	0.03191	0.09091	OK
8	4000	25.85	290827.79	9034.16	0.03783	0.09091	OK
7	4000	28.13	328575.83	9592.81	0.04380	0.09091	OK

6	4000	30.28	366323.87	10180.06	0.04953	0.09091	OK
5	4000	32.17	404071.90	10794.90	0.05473	0.09091	OK
4	4000	33.39	441819.94	11414.69	0.05874	0.09091	OK
3	4000	33.00	479567.98	11971.47	0.06009	0.09091	OK
2	4000	28.68	517316.02	12397.65	0.05440	0.09091	OK
1	4000	14.36	555657.53	12591.01	0.02880	0.09091	OK

From the table above, it is found that the stability coefficients of each story in the Y direction with load case RSY do not surpass the maximum value of 0.09091. Finally, after checking the stability coefficient values compared to the maximum coefficient in both directions, the building can be determined as safe according to the SNI 1726:2019 P-delta effect criteria.

To visualize the results, the stability coefficient values are depicted into a graph for each direction.

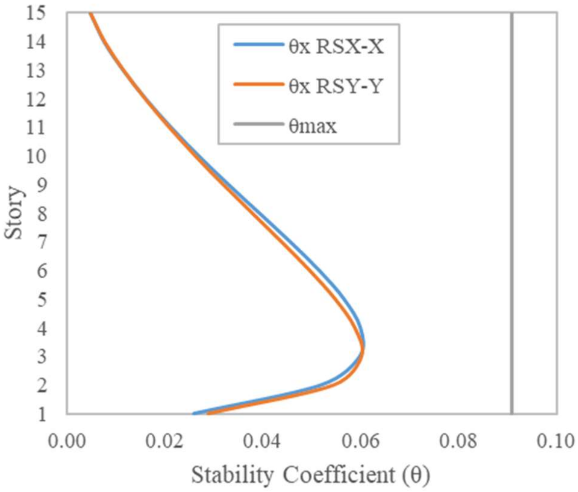


Figure 5.9 Stage 2 RS P-Delta Effect Comparison in X and Y Directions

From the figure, it can be concluded that the stability coefficient in the analysis of P-delta effect in X and Y directions are not extremely distinctive, and that both have fulfilled the requirements of the maximum coefficient value.

With suitable story drift, P-delta effect, and member internal forces, the design process can continue to the foundation design. Moreover, the process continues to analyze spring stiffness for the flexible foundation.

5.6 Structural Member Design

5.6.1 Main Beam Reinforcement Design

According to SNI 2847:2019 Article 18.6.2, the dimension of the beam must fulfill the following requirements. As the dimension of BI1X and BI1Y is the same (900 mm of height and 450 mm of width), the analysis results are the same.

1. $\mathrm{Ln} \geq 4\mathrm{D}_{\mathrm{flexural}}$

   $8000 - 2\left(\frac{1}{2} \cdot \mathrm{B}_{\mathrm{column}}\right) \geq 4(28)$

   $7000\ \mathrm{mm} > 112\ \mathrm{mm}$ (OK)

2. $\mathrm{B} \geq 0.3\mathrm{H}$ or $\mathrm{B} \geq 250\ \mathrm{mm}$ (the smaller value)

   With $0.3\mathrm{H} = 300\ \mathrm{mm}$ and $\mathrm{B} = 450\ \mathrm{mm}$, the smaller value is 250 mm.

   $450\ \mathrm{mm} > 250\ \mathrm{mm}$ (OK)

Before designing the reinforcement of the main beams, the support area and middle span area must be determined beforehand. The support area starts from the edge of the beam until a quarter (1/4) of the beam length on both sides, while the middle part of the beam is called the middle span area.

The ultimate moments (Mu) of the main beams are obtained from the ETABS model. These ultimate moments are then checked to be redistributed. To check if the moment needs redistribution, the percentage of positive moments against negative moments in the support area is calculated. Meanwhile, for the middle span area, the percentage of negative moments against positive moments is calculated. If the percentage shows a value larger than or equal to 50%, the moment does not need to be redistributed. On the other hand, if the percentage is less than 50%, the moment needs to be redistributed. The moment redistribution checking is as follows.

Table 5.37 BI1X Moment Redistribution Checking

Beam	Story	Support				Middle Span			
		M+	M-	Moment (%)	Status	M+	M-	Moment (%)	Status
BI1X	1	222.96	-512.77	43.48	Redis	214.78	-84.28	39.24	Redis
	2,3,4,5	423.64	-681.20	62.19	OK	286.46	-171.11	59.73	OK
	6,7,8	411.34	-656.28	62.68	OK	283.39	-166.89	58.89	OK
	9,10,11	336.10	-587.03	57.25	OK	257.13	-134.25	52.21	OK
	12,13,14	218.73	-447.97	48.83	Redis	207.00	-73.74	35.62	Redis
	15	129.67	-237.84	54.52	OK	133.65	-26.92	20.14	Redis

Table 5.38 BI1Y Moment Redistribution Checking

Beam	Story	Support				Middle Span			
		M+	M-	Moment (%)	Status	M+	M-	Moment (%)	Status
BI1Y	1	327.27	-529.85	61.77	OK	238.09	-120.17	50.47	OK
	2,3,4,5	532.84	-734.26	72.57	OK	302.04	-207.74	68.78	OK
	6,7,8	520.59	-724.74	71.83	OK	298.11	-201.40	67.56	OK
	9,10,11	438.41	-646.53	67.81	OK	268.16	-166.47	62.08	OK
	12,13,14	274.62	-486.21	56.48	OK	207.37	-98.73	47.61	Redis
	15	97.90	-256.93	38.10	Redis	157.85	-32.60	20.65	Redis

From the tables above, it is found that the moments of beams BI1X and BI1Y need redistribution in some of the upper stories. The moments that need redistribution are then analyzed as follows.

Table 5.39 Moment Redistribution Analysis of Main Beam BI1X

Support Area							
Story	% Redistribution	% Red. x M-	M-	ΣM	M+	Moment (%)	Status
1	5	25.64	487.13	1471.45	248.60	51.03	OK
2,3,4,5	0	0	681.20	2209.67	423.64	62.19	OK
6,7,8	0	0	656.28	2135.24	411.34	62.68	OK
9,10,11	0	0	587.03	1846.27	336.10	57.25	OK
12,13,14	1	4.48	443.49	1333.39	223.21	50.33	OK
15	0	0	237.84	735.03	129.67	54.52	OK
Middle Span Area							
Story	% Redistribution	% Red. x M+	M-	ΣM	M+	Moment (%)	Status
1	8	17.18	197.59	598.11	101.46	51.35	OK
2,3,4,5	0	0	286.46	915.13	171.11	59.73	OK
6,7,8	0	0	283.39	900.54	166.89	58.89	OK
9,10,11	0	0	257.13	782.75	134.25	52.21	OK
12,13,14	10	20.70	186.30	561.47	94.44	50.69	OK
15	20	26.73	106.92	321.14	53.65	50.18	OK

Table 5.40 Moment Redistribution Analysis of Main Beam BI1Y

Support Area							
Story	% Redistribution	% Red. x M-	M-	ΣM	M+	Moment (%)	Status
1	0	0	529.85	1714.24	327.27	61.77	OK
2,3,4,5	0	0	734.26	2534.20	532.84	72.57	OK
6,7,8	0	0	724.74	2490.66	520.59	71.83	OK
9,10,11	0	0	646.53	2169.88	438.41	67.81	OK
12,13,14	0	0	486.21	1521.64	274.62	56.48	OK
15	8	20.55	236.38	709.66	118.45	50.11	OK
Middle Span Area							
Story	% Redistribution	% Red. x M+	M-	ΣM	M+	Moment (%)	Status
1	0	0	238.09	716.53	120.17	50.47	OK
2,3,4,5	0	0	302.04	1019.56	207.74	68.78	OK
6,7,8	0	0	298.11	999.03	201.40	67.56	OK
9,10,11	0	0	268.16	869.26	166.47	62.08	OK
12,13,14	2	4.15	203.22	612.20	102.88	50.62	OK
15	20	31.57	126.28	380.90	64.17	50.82	OK

From the tables above, it can be concluded that the moments are safe after being redistributed. Hence, the final redistributed ultimate moments are as follows.

Table 5.41 Final Redistributed Ultimate Moments of Main Beam BI1X

BI1X	Support		Middle Span	
Story	M+	M-	M+	M-
1	248.60	487.13	197.59	101.46
2,3,4,5	423.64	681.20	286.46	171.11
6,7,8	411.34	656.28	283.39	166.89
9,10,11	336.10	587.03	257.13	134.25
12,13,14	223.21	443.49	186.30	94.44
15	129.67	237.84	106.92	53.65

Table 5.42 Final Redistributed Ultimate Moments of Main Beam BI1Y

BI1Y	Support		Middle Span	
Story	M+	M-	M+	M-
1	327.27	529.85	238.09	120.17
2,3,4,5	532.84	734.26	302.04	207.74
6,7,8	520.59	724.74	298.11	201.40
9,10,11	438.41	646.53	268.16	166.47
12,13,14	274.62	486.21	203.22	102.88
15	118.45	236.38	126.28	64.17

The next step is to design the reinforcement of the main beams BI1X and BI1Y in each story. The reinforcement design example of the main beam BI1X support area in the first story is as follows.

Mu- = 487.13 kN-m

Mu+ = 248.60 kN-m

The material properties are as follows.

φ = 0.9

f'c = 35 MPa

Ɛc = 0.003

β = 0.80

fy = 400 MPa

E = 200000 MPa

Ɛy = 0.002

Meanwhile, the dimensions and details of the main beam BI1X support area are as follows.

Dp (flexural)	= 28 mm
Ds (shear)	= 10 mm
H	= 900 mm
B	= 450 mm
Concrete cover	= 40 mm
Reinforcement spacing (s)	= 25 mm
ds = ds'	= 64 mm
d = d'	= 836 mm

The assumption of the number of reinforcements needed is analyzed as follows.

1. Tensile moment

$\mathrm{Mn_1} = \varphi \cdot \mathrm{Mu^-} \cdot \mathrm{R} = 0.9 \cdot 487.13 \cdot 0.5$

$\mathrm{Mn_1} = 219.21$ kN-m $= 219207636$ N-mm

2. Quadratic formula to determine a value

Using the following quadratic formula to determine a value:

$\mathrm{Mn} = 0.85 \cdot \mathrm{f'c} \cdot \mathrm{a} \cdot \mathrm{b} \cdot \left(\mathrm{d} - \frac{\mathrm{a}}{2}\right)$

$219207636 = 0.85 \cdot 35 \cdot \mathrm{a} \cdot 450 \cdot \left(836 - \frac{\mathrm{a}}{2}\right)$

$219207636 = 11191950\mathrm{a} - 6693.75\mathrm{a}^2$

$6693.75a^2 - 11191950a + 219207636 = 0$

From the quadratic formula, the value of a is obtained 19.82 mm.

3. Compressive and tensile area

$As_1 = \frac{0.85 \cdot f\prime c \cdot a \cdot b}{fy} = \frac{0.85 \cdot 35 \cdot 19.82 \cdot 450}{400} = 663.39 \text{ mm}^2$

$Mn_2 = Mu^- - Mn_1 = 487.13 - 219.21$

$Mn_2 = 267.92$ kN-m $= 267920444$ N-mm

$Ts_2 = \frac{Mn_2}{fy} = \frac{267920444}{400} = 669801.11$ N

$As_2 = \frac{Ts_2}{fy} = \frac{669801.11}{400} = 1674.50 \text{ mm}^2$

$As_{tensile} = As_1 + As_2 = 663.39 + 1674.50 = 2337.89 \text{ mm}^2$

$As_{compression} = As_2 = 1674.50 \text{ mm}^2$

$As_{1D} = \frac{1}{4}\pi \cdot Dp^2 = \frac{1}{4}\pi \cdot 28^2 = 615.75 \text{ mm}^2$

4. Number of reinforcements (n)

$n_{upper} = \frac{As_{tensile}}{As_{1D}} = \frac{2337.89}{615.75} = 3.80 \approx 4$

$n_{lower} = \frac{As_{compression}}{As_{1D}} = \frac{1674.50}{615.75} = 2.72 \approx 3$

Check spacing:

$s = \frac{B-(2 \cdot \text{Concrete cover})-(2 \cdot Ds)-(n \cdot Dp)}{n-1} = 79.33 \text{ mm} > 25 \text{ mm (OK)}$

Hence, the number of reinforcements for main beam BI1X in support area is obtained 4 in the upper (tensile) area and 3 in the lower (compression) area.

The analysis is then continued to determine the available or nominal moments. The analysis of the negative nominal moment of main beam BI1X in support area is as follows.

1. Area and reinforcement

$As^- = n_{upper} \cdot As_{1D} = 4 \cdot 615.75 \text{ mm}^2 = 2463.01 \text{ mm}^2$

$As^+ = n_{lower} \cdot As_{1D} = 3 \cdot 615.75 \text{ mm}^2 = 1847.26 \text{ mm}^2$

$ds = ds' = 64$ mm

$d = d' = 836$ mm

2. Reinforcement condition assumption

Tensile area = yielded

Compression area = not yet yielded

3. Quadratic formula to determine c value

Using the following quadratic formula to determine c value:

$$As^{-} \cdot fy = 0.85 \cdot f'c \cdot \beta \cdot b + \left(\frac{c-ds'}{c}\right) \cdot \varepsilon c \cdot Es \cdot As^{+}$$

$$985203.46c = 10710c^2 - 70934648.84 + 1108353.89c$$

$$10710c^2 + 123150.43c - 70934648.84 = 0$$

From the quadratic formula, the value of c is obtained 75.84 mm.

Meanwhile, the value of a is calculated as follows.

$$a = c \cdot \beta = 75.84 \cdot 0.80 = 60.67 \text{ mm}$$

4. Value of fs

$$fs = \left(\frac{c-ds'}{c}\right) \cdot \varepsilon c \cdot Es = \left(\frac{75.84-64}{75.84}\right) \cdot 0.003 \cdot 200000 = 93.65 \text{ MPa}$$

5. Negative moment (M^-)

$$Mn_{cc} = 0.85 \cdot f'c \cdot a \cdot b \cdot \left(d - \frac{a}{2}\right) = 0.85 \cdot 35 \cdot 60.67 \cdot 450 \cdot \left(836 - \frac{60.67}{2}\right)$$

$$Mn_{cc} = 654369902.01 \text{ N-mm}$$

$$Mn_{cs} = As^{+} \cdot fs \cdot (d - ds') = 1847.26 \cdot 93.65 \cdot (836 - 64)$$

$$Mn_{cs} = 133550509.66 \text{ N-mm}$$

$$Mn = Mn_{cc} + Mn_{cs} = 654369902.01 + 133550509.66$$

$$Mn = 787920411.67 \text{ N-mm} = 787.92 \text{ kN-m}$$

$$\varphi Mn = 0.9 \cdot 787.92 = 709.13 \text{ kN-m}$$

Check towards Mu:

$$\varphi Mn > Mu^{-}$$

$709.13 > 487.13$ (SAFE)

Meanwhile, the analysis of the positive nominal moment of main beam BI1X in support area is as follows.

1. Area and reinforcement

$$As^{+} = n_{lower} \cdot As_{1D} = 3 \cdot 615.75 \text{ mm}^2 = 1847.26 \text{ mm}^2$$

$$As^{-} = n_{upper} \cdot As_{1D} = 4 \cdot 615.75 \text{ mm}^2 = 2463.01 \text{ mm}^2$$

$$ds = ds' = 64 \text{ mm}$$

$$d = d' = 836 \text{ mm}$$

2. Reinforcement condition assumption

Tensile area = yielded

Compression area = not yet yielded

3. Quadratic formula to determine c value

Using the following quadratic formula to determine c value:

$$\mathrm{As}^{+} \cdot \mathrm{fy} = 0.85 \cdot \mathrm{f'c} \cdot \beta \cdot \mathrm{b} + \left(\frac{\mathrm{c-ds}}{\mathrm{c}}\right) \cdot \varepsilon\mathrm{c} \cdot \mathrm{Es} \cdot \mathrm{As}^{-}$$

$$738902.59\mathrm{c} = 10710\mathrm{c}^2 - 94579531.79 + 1477805\mathrm{c}$$

$$10710\mathrm{c}^2 + 738902.59\mathrm{c} - 94579531.79 = 0$$

From the quadratic formula, the value of c is obtained 65.61 mm.

Meanwhile, the value of a is calculated as follows.

$$\mathrm{a} = \mathrm{c} \cdot \beta = 65.61 \cdot 0.80 = 52.49 \text{ mm}$$

4. Value of fs

$$\mathrm{fs} = \left(\frac{\mathrm{c-ds}}{\mathrm{c}}\right) \cdot \varepsilon\mathrm{c} \cdot \mathrm{Es} = \left(\frac{65.61-6}{65.61}\right) \cdot 0.003 \cdot 200000 = 14.71 \text{ MPa}$$

5. Positive moment (M^{+})

$$\mathrm{Mn_{cc}} = 0.85 \cdot \mathrm{f'c} \cdot \mathrm{a} \cdot \mathrm{b} \cdot \left(\mathrm{d'} - \frac{\mathrm{a}}{2}\right) = 0.85 \cdot 35 \cdot 52.49 \cdot 450 \cdot \left(836 - \frac{52.49}{2}\right)$$

$$\mathrm{Mn_{cc}} = 568990485.94 \text{ N-mm}$$

$$\mathrm{Mn_{cs}} = \mathrm{As}^{-} \cdot \mathrm{fs} \cdot (\mathrm{d'} - \mathrm{ds}) = 2463.01 \cdot 14.71 \cdot (836 - 64)$$

$$\mathrm{Mn_{cs}} = 27972651.24 \text{ N-mm}$$

$$\mathrm{Mn} = \mathrm{Mn_{cc}} + \mathrm{Mn_{cs}} = 568990485.94 + 27972651.24$$

$$\mathrm{Mn} = 596963137.17 \text{ N-mm} = 596.96 \text{ kN-m}$$

$$\varphi\mathrm{Mn} = 0.9 \cdot 596.96 = 537.27 \text{ kN-m}$$

Check towards Mu:

$$\varphi\mathrm{Mn} > \mathrm{Mu}^{+}$$

$$537.27 > 248.60 \text{ (SAFE)}$$

The analysis is then continued to determine the probable moments (Mpr). The analysis of the negative probable moment of main beam BI1X in support area is as follows.

1. Area and reinforcement

$$\mathrm{As_{tensile}} = \mathrm{n_{upper}} \cdot \mathrm{As_{1D}} = 4 \cdot 615.75 \text{ mm}^2 = 2463.01 \text{ mm}^2$$

$As_{compression} = n_{lower} \cdot As_{1D} = 3 \cdot 615.75 \text{ mm}^2 = 1847.26 \text{ mm}^2$

$ds = ds' = 64 \text{ mm}$

$d = d' = 836 \text{ mm}$

$ø_{os} = 1.25$

2. Reinforcement condition assumption

   Tensile area = yielded

   Compression area = not yet yielded

3. Quadratic formula to determine c value

   Using the following quadratic formula to determine c value:

   $ø_{os} \cdot As_{tensile} \cdot fy = 0.85 \cdot f'c \cdot \beta \cdot b + \left(\frac{c-ds'}{c}\right) \cdot \varepsilon c \cdot Es \cdot As_{compression}$

   $1231504.32c = 10710c^2 - 70934648.84 + 1108353.89c$

   $10710c^2 - 123150.43c - 70934648.84 = 0$

   From the quadratic formula, the value of c is obtained 87.34 mm.

   Meanwhile, the value of a is calculated as follows.

   $a = c \cdot \beta = 87.34 \cdot 0.80 = 69.87 \text{ mm}$

4. Value of fs

   $fs = \left(\frac{c-ds'}{c}\right) \cdot \varepsilon c \cdot Es = \left(\frac{87.34-64}{87.34}\right) \cdot 0.003 \cdot 200000 = 160.32 \text{ MPa}$

5. Check Ts = Cc + Cs

   $Cc = 0.85 \cdot f'c \cdot a \cdot b = 0.85 \cdot 35 \cdot 69.87 \cdot 450 = 935361 \text{ N}$

   $Cs = As_{compression} \cdot fs = 1847.26 \cdot 160.32 = 296143.32 \text{ N}$

   $Ts = ø_{os} \cdot As_{tensile} \cdot fy = 1.25 \cdot 2463.01 \cdot 400 = 1231504.32 \text{ N}$

   Check Ts = Cc + Cs:

   $Ts = Cc + Cs$

   $1231504.32 = 935361 + 296143.32$

   $1231504.32 = 1231504.32$ (OK)

6. Tensile probable moment (Mpr⁻)

   $Mn_{cc} = 0.85 \cdot f'c \cdot a \cdot b \cdot \left(d - \frac{a}{2}\right) = 0.85 \cdot 35 \cdot 69.87 \cdot 450 \cdot \left(836 - \frac{69.87}{2}\right)$

   $Mn_{cc} = 749285784.80 \text{ N-mm}$

   $Mn_{cs} = As_{compression} \cdot fs \cdot (d - ds') = 1847.26 \cdot 160.32 \cdot (836 - 64)$

$\text{Mn}_{\text{cs}} = 228622643.18$ N-mm

$\text{Mn} = \text{Mn}_{\text{cc}} + \text{Mn}_{\text{cs}} = 749285784.80 + 228622643.18$

$\text{Mn} = 977908427.98$ N-mm $= 977.91$ kN-m

$\varphi\text{Mn} = 0.9 \cdot 977.91 = 880.12$ kN-m

Meanwhile, the analysis of the positive probable moment of main beam BI1X in support area is as follows.

1. Area and reinforcement

$\text{As}_{\text{compression}} = \text{n}_{\text{lower}} \cdot \text{As}_{\text{1D}} = 3 \cdot 615.75\ \text{mm}^2 = 1847.26\ \text{mm}^2$

$\text{As}_{\text{tensile}} = \text{n}_{\text{upper}} \cdot \text{As}_{\text{1D}} = 4 \cdot 615.75\ \text{mm}^2 = 2463.01\ \text{mm}^2$

$\text{ds} = \text{ds}' = 64\ \text{mm}$

$\text{d} = \text{d}' = 836\ \text{mm}$

$ø_{\text{os}} = 1.25$

2. Reinforcement condition assumption

Tensile area = yielded

Compression area = not yet yielded

3. Quadratic formula to determine c value

Using the following quadratic formula to determine c value:

$ø_{\text{os}} \cdot \text{As}_{\text{compression}} \cdot \text{fy} = 0.85 \cdot \text{f}'\text{c} \cdot \beta \cdot \text{b} + \left(\frac{\text{c}-\text{ds}}{\text{c}}\right) \cdot \varepsilon\text{c} \cdot \text{Es} \cdot \text{As}_{\text{tensile}}$

$923628.24\text{c} = 10710\text{c}^2 - 94579531.79 + 1477805.18\text{c}$

$10710\text{c}^2 + 554176.94\text{c} - 94579531.79 = 0$

From the quadratic formula, the value of c is obtained 71.60 mm.

Meanwhile, the value of a is calculated as follows.

$\text{a} = \text{c} \cdot \beta = 71.60 \cdot 0.80 = 57.28\ \text{mm}$

4. Value of fs

$\text{fs} = \left(\frac{\text{c}-\text{ds}}{\text{c}}\right) \cdot \varepsilon\text{c} \cdot \text{Es} = \left(\frac{71.60-6}{71.60}\right) \cdot 0.003 \cdot 200000 = 63.67\ \text{MPa}$

5. Check Ts = Cc + Cs

$\text{Cc} = 0.85 \cdot \text{f}'\text{c} \cdot \text{a} \cdot \text{b} = 0.85 \cdot 35 \cdot 57.28 \cdot 450 = 766810.38\ \text{N}$

$\text{Cs} = \text{As}_{\text{tensile}} \cdot \text{fs} = 1847.26 \cdot 63.67 = 156817.86\ \text{N}$

$\text{Ts} = ø_{\text{os}} \cdot \text{As}_{\text{compression}} \cdot \text{fy} = 1.25 \cdot 1847.26 \cdot 400 = 923628.24\ \text{N}$

Check Ts = Cc + Cs:

$\mathrm{Ts} = \mathrm{Cc} + \mathrm{Cs}$

$923628.24\ = 766810.38 + 156817.86$

$923628.24\ = 923628.24$ (OK)

6. Compression probable moment (Mpr^{+})

$\mathrm{Mn_{cc}} = 0.85 \cdot \mathrm{f'c} \cdot \mathrm{a} \cdot \mathrm{b} \cdot \left(\mathrm{d'} - \frac{\mathrm{a}}{2}\right) = 0.85 \cdot 35 \cdot 57.28 \cdot 450 \cdot \left(836 - \frac{57.28}{2}\right)$

$\mathrm{Mn_{cc}} = 619092759.72$ N-mm

$\mathrm{Mn_{cs}} = \mathrm{As_{tensile}} \cdot \mathrm{fs} \cdot (\mathrm{d'} - \mathrm{ds}) = 2463.01 \cdot 63.67 \cdot (836 - 64)$

$\mathrm{Mn_{cs}} = 121063390.43$ N-mm

$\mathrm{Mn} = \mathrm{Mn_{cc}} + \mathrm{Mn_{cs}} = 619092759.72 + 121063390.43$

$\mathrm{Mn} = 740156150.15$ N-mm $= 740.16$ kN-m

$\varphi\mathrm{Mn} = 0.9 \cdot 740.16 = 666.14$ kN-m

Finally, all the results of the number of reinforcement (n), nominal and probable moments of main beam BI1X in support area are recapitulated in the following table.

Table 5.43 Moment and Flexural Reinforcement Results of BI1X Support Area

BI1X Support Area		
Mu-	487.13	kNm
Mu+	248.60	kNm
M-	709.13	kNm
M+	537.27	kNm
Mpr-	880.12	kNm
Mpr+	666.14	kNm
Upper Reinforcement	4	piece
	4D28	
Lower Reinforcement	3	piece
	3D28	

The number of reinforcement (n) and moment analysis are then conducted for the middle span with the same steps. The difference between support and middle span area is as follows.

1. Support area: Upper reinforcement area is the negative or tensile area, and lower reinforcement area is the positive or compression area.

2. Middle span area: Upper reinforcement area is the positive or compression area, and lower reinforcement area is the negative or tensile area.

The results of the number of reinforcement (n), nominal and probable moments of main beam BI1X in middle span area are recapitulated in the following table.

Table 5.44 Moment and Flexural Reinforcement Results of BI1X Middle Span Area

BI1X Middle Span Area		
Mu-	101.46	kNm
Mu+	197.59	kNm
M-	364.70	kNm
M+	339.07	kNm
Mpr-	451.10	kNm
Mpr+	451.10	kNm
Upper Reinforcement	2	piece
	2D28	
Lower Reinforcement	2	piece
	2D28	

This analysis is conducted on both main beams BI1X and BI1Y in all the story groups. The results are as follows.

Table 5.45 Recapitulation of Moment and Flexural Reinforcement Results of BI1X

Beam	Description	Story												Unit
		1		2,3,4,5		6,7,8		9,10,11		12,13,14		15		
		Support	Middle Span	Support	Middle Span	Support	Middle Span	Support	Middle Span	Support	Middle Span	Support	Middle Span	
BI1X	Mu-	487.13	101.46	681.20	171.11	656.28	166.89	587.03	134.25	443.49	94.44	237.84	53.65	kNm
	Mu+	248.60	197.59	423.64	286.46	411.34	283.39	336.10	257.13	223.21	186.30	129.67	106.92	kNm
	M-	709.13	364.70	1050.70	537.30	1050.70	537.30	880.29	364.70	709.13	364.70	364.70	364.70	kNm
	M+	537.27	339.07	708.89	507.19	708.89	507.19	708.96	339.07	537.27	339.07	364.70	339.07	kNm
	Mpr-	880.12	451.10	1304.00	666.24	1304.00	666.24	1093.14	451.10	880.12	451.10	451.10	451.10	kNm
	Mpr+	666.14	451.10	880.31	666.24	880.31	666.24	880.32	451.10	666.14	451.10	451.10	451.10	kNm
	Upper reinforcement	4	2	6	3	6	3	5	2	4	2	2	2	piece
		4D28	2D28	6D28	3D28	6D28	3D28	5D28	2D28	4D28	2D28	2D28	2D28	
	Lower reinforcement	3	2	4	3	4	3	4	2	3	2	2	2	piece
		3D28	2D28	4D28	3D28	4D28	3D28	4D28	2D28	3D28	2D28	2D28	2D28	

Table 5.46 Recapitulation of Moment and Flexural Reinforcement Results of BI1Y

Beam	Description	Story												Unit
		1		2,3,4,5		6,7,8		9,10,11		12,13,14		15		
		Support	Middle Span	Support	Middle Span	Support	Middle Span	Support	Middle Span	Support	Middle Span	Support	Middle Span	
BI1Y	Mu-	529.85	120.17	734.26	207.74	724.74	201.40	646.53	166.47	486.21	102.88	236.38	64.17	kNm
	Mu+	327.27	238.09	532.84	302.04	520.59	298.11	438.41	268.16	274.62	203.22	118.45	126.28	kNm
	M-	880.12	364.70	1051.13	537.30	1051.13	537.30	1050.70	537.30	709.13	364.70	364.70	364.70	kNm
	M+	537.24	339.07	880.31	507.19	880.31	507.19	708.89	507.19	537.27	339.07	364.70	339.07	kNm
	Mpr-	1092.02	451.10	1305.73	666.24	1305.73	666.24	1304.00	666.24	880.12	451.10	451.10	451.10	kNm
	Mpr+	666.06	451.10	1094.01	666.24	1094.01	666.24	880.31	666.24	666.14	451.10	451.10	451.10	kNm
	Upper reinforcement	5	2	6	3	6	3	6	3	4	2	2	2	piece
		5D28	2D28	6D28	3D28	6D28	3D28	6D28	3D28	4D28	2D28	2D28	2D28	
	Lower reinforcement	3	2	5	3	5	3	4	3	3	2	2	2	piece
		3D28	2D28	5D28	3D28	5D28	3D28	4D28	3D28	3D28	2D28	2D28	2D28	

For the shear reinforcement design of main beams, the shear force values obtained from the analysis of the ETABS model due to gravitational load (Vg) within and outside the plastic joint area (Lo) can be seen in the following table.

Table 5.47 ETABS Shear Force Values for Shear Reinforcement Design of Main Beams

Beam	Story	PJ	≠PJ	PJ
		Vg left	Vg upper/lower	Vg right
BI1X	1	-236.73	194.60	236.73
	2,3,4,5	-212.36	167.06	212.36
	6,7,8	-201.44	156.79	201.44
	9,10,11	-195.80	151.50	195.80
	12,13,14	-194.66	150.93	194.66
	15	-134.38	104.59	134.38
BI1Y	1	-230.76	177.89	230.76
	2,3,4,5	-219.51	169.25	219.51
	6,7,8	-226.33	175.78	226.33
	9,10,11	-230.74	180.02	230.74
	12,13,14	-233.67	182.82	233.67
	15	-157.27	125.92	157.27

The initial data of main beam BI1X material properties for shear reinforcement design in the first story is as follows.

L_{beam} = 8 m = 8000 mm

$B_{left\ column}$ = 1000 mm

$B_{right\ column}$ = 1000 mm

L_{netto} = 8000 – 0.5(1000) – 0.5(1000) = 7000 mm = 7 m

Mpr^{-} = 666.24 kNm

Mpr^{+} = 666.24 kNm

B_{beam} = 450 mm

H_{beam} = 900 mm

f’c = 35 MPa

$fy_{stirrup}$ = 360 MPa

D_{shear} = 10 mm

φ = 0.75

H^{-} = 900 mm

$D_{flexural}$ = 28 mm

The shear force analysis to determine the shear reinforcement (stirrup) of the main beam BI1X in the first story as an example is as follows.

1. Shear force due to gravitational load (Vg)

Vg_{left} = 236.73 kN → Vg_{left}/φ = 236.77/0.75 = 315.64 kN

Vg_{right} = 236.73 kN → Vg_{left}/φ = -(236.73/0.75) = -315.64 kN

Vg_{upper} = 194.60 kN → Vg_{upper}/φ = 194.60/0.75 = 259.47 kN

Vg_{lower} = 194.60 kN → Vg_{lower}/φ = -(194.60/0.75) = -259.47 kN

The shear force diagram (SFD) obtained from the Vg values is as follows.

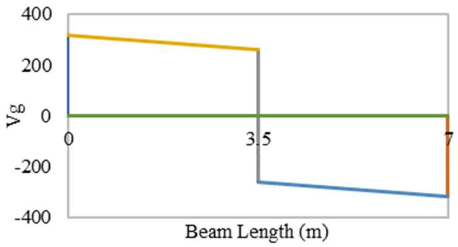


Figure 5.10 SFD of Vg in BI1X Story 1

2. Shear force due to earthquake load (Ve)

Earthquake direction is taken from the left:

$$Ve_{left} = -\left(\frac{Mpr^{+}}{L_{netto}\cdot\varphi} + \frac{Mpr^{-}}{L_{netto}\cdot\varphi}\right) = -\left(\frac{666.24}{7\cdot 0.75} + \frac{666.24}{7\cdot 0.75}\right) = -253.80\ \text{kN}$$

$$Ve_{right} = \left(\frac{Mpr^{-}}{L_{netto}\cdot\varphi} + \frac{Mpr^{+}}{L_{netto}\cdot\varphi}\right) = \left(\frac{666.24}{7\cdot 0.75} + \frac{666.24}{7\cdot 0.75}\right) = 253.80\ \text{kN}$$

Because the earthquake direction is from the left, the Ve is taken -253.80 kN. Meanwhile, the shear force diagram (SFD) obtained from the Ve values is as follows.

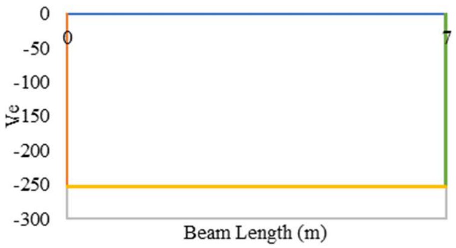


Figure 5.11 SFD of Ve in BI1X Story 1

3. Ultimate shear force (Vu) combination of Vg and Ve

$Vu_{left} = Vg_{left} + Ve = 315.64 + |-253.80| = 569.45$ kN

$Vu_{right} = Vg_{right} + Ve = (-315.64) + (-253.80) = -569.45$ kN

$Vu_{upper} = Vg_{upper} + Ve = 259.47 + |-253.80| = 513.27$ kN

$Vu_{lower} = Vg_{lower} + Ve = (-259.47) + (-253.80) = -513.27$ kN

The shear force diagram (SFD) obtained from the Vu combination values is as follows.

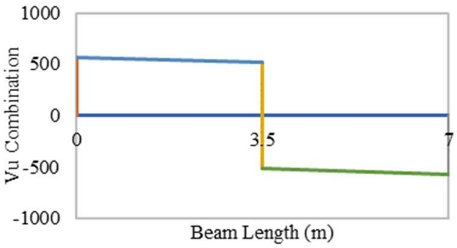


Figure 5.12 SFD of Vu Combination in BI1X Story 1

4. Diagram dimension

The dimensions of the shear force diagrams are determined as follows.

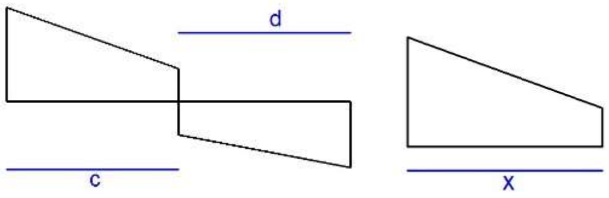


Figure 5.13 Illustration of the SFD Dimension

$$c = \frac{Vu_{left} - Vu_{upper}}{(Vu_{left} - Vu_{upper}) + (Vu_{right} - Vu_{lower})} \cdot L_{netto}$$

$$c = \frac{569.45 - 51\quad .27}{(569.45 - 5\quad .27) + |(-569.45) - (-513.27)|} \cdot 7000 = 3500\ mm = 3.5\ m$$

$$d = L_{netto} - c = 7000 - 3500 = 3500\ mm = 3.5\ m$$

x is the bigger value between c and d, so the value is concluded as c = 3500 mm.

5. Plastic joint area

$\mathrm{Vc} = \frac{1}{6} \cdot \sqrt{\mathrm{f'c}} \cdot \mathrm{B_{beam}} \cdot \mathrm{H^-} = \frac{1}{6} \cdot \sqrt{35} \cdot 450 \cdot 900$

$\mathrm{Vc} = 399335.39\ \mathrm{N} = 399.34\ \mathrm{kN}$

If the value of Ve > $\mathrm{Vg_{right}}$, the $\mathrm{Vs_1}$ value is determined as the bigger value between $\mathrm{Vu_{left}}$ and $\mathrm{Vu_{right}}$. Meanwhile, if it is the other way around, the $\mathrm{Vs_1}$ value is subtracted by the value of Vc.

$\mathrm{Ve} < \mathrm{Vg_{right}}$

$253.80\ \mathrm{kN} < 315.64\ \mathrm{kN}$

$\mathrm{Vs_1} = \mathrm{Vu_{left}} - \mathrm{Vc} = 569.45 - 399.34 = 170.11\ \mathrm{kN}$

$\mathrm{Av} = \frac{1}{4} \cdot \pi \cdot \mathrm{D_{shear}}^2 = \frac{1}{4} \cdot \pi \cdot 10^2 = 78.54\ \mathrm{mm^2}$

$\mathrm{n_{stirrup}} = 2$

$\mathrm{fy_{stirrup}} = 360\ \mathrm{N/mm^2} = 0.36\ \mathrm{kN/mm^2}$

$\mathrm{s} = \mathrm{n_{stirrup}} \cdot \frac{\mathrm{Av \cdot fy_{stirrup} \cdot H^-}}{\mathrm{Vs_1}} = 2 \cdot \frac{78.54 \cdot 0.36 \cdot 900}{170.11} = 299.18\ \mathrm{mm}$

$\mathrm{s_{used}} = 85\ \mathrm{mm}$

Check:

a. $\frac{\mathrm{H^-}}{4} = \frac{900}{4} = 225\ \mathrm{mm}$

b. $8 \cdot \mathrm{D_{flexural}} = 8 \cdot 28 = 224\ \mathrm{mm}$

c. $24 \cdot \mathrm{D_{shear}} = 24 \cdot 10 = 240\ \mathrm{mm}$

d. $300\ \mathrm{mm}$

The minimum value of the requirements above is 224 mm.

$50\ \mathrm{mm} \leq \mathrm{s_{used}} \leq 224\ \mathrm{mm}$

$50\ \mathrm{mm} < 85\ \mathrm{mm} < 224\ \mathrm{mm}$ (OK)

Hence, the shear reinforcement (stirrup) used in the plastic joint area is 2P10-85 mm.

6. Outside plastic joint area

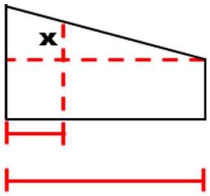


Figure 5.14 Illustration of the SFD for the Outside of Plastic Joint Area Analysis

$V_{ats} = Vu_{left} - Ve - Vg_{upper}$

$= 569.45 - 253.80 - 259.47$

$= 56.18\ kN$

$x = \frac{V_{ats} \cdot (x - H_{beam})}{x}$

$= \frac{56.18 \cdot (3500 - 900)}{3500} = 27.29\ mm$

$y = x + Vg_{upper} + Ve$

$= 27.29 + 259.47 + 253.80 = 540.56\ kN$

$Vs_2 = y - Vc = 540.56 - 399.34 = 141.22\ kN$

$n_{stirrup} = 2$

$fy_{stirrup} = 360\ N/mm^2 = 0.36\ kN/mm^2$

$s = n_{stirrup} \cdot \frac{Av \cdot fy_{stirrup} \cdot H^-}{Vs_2} = 2 \cdot \frac{78.54 \cdot 0.36 \cdot 900}{141.22} = 360.39\ mm$

$s_{used} = 150\ mm$

Check:

$50\ mm \leq s_{used} \leq \frac{H^-}{2}$

$50\ mm \leq 150\ mm \leq \frac{900}{2}$

$50\ mm < 150\ mm < 450\ mm$ (OK)

Hence, the shear reinforcement (stirrup) used outside the plastic joint area is 2P10-150 mm.

Furthermore, the shear reinforcement design is conducted on both main beams BI1X and BI1Y in all the story groups. The results are as follows.

Table 5.48 Recapitulation Shear Reinforcement Results of BI1X

Beam	Story	Stirrup	
		Plastic Joint	Outside Plastic Joint
BI1X	1	2P10-85 mm	2P10-150 mm
	2,3,4,5	2P10-85 mm	2P10-150 mm
	6,7,8	2P10-85 mm	2P10-150 mm
	9,10,11	2P10-85 mm	2P10-150 mm
	12,13,14	2P10-85 mm	2P10-150 mm
	15	2P10-85 mm	2P10-150 mm

Table 5.49 Recapitulation Shear Reinforcement Results of BI1Y

Beam	Story	Stirrup	
		Plastic Joint	Outside Plastic Joint
BI1Y	1	2P10-85 mm	2P10-150 mm
	2,3,4,5	2P10-85 mm	2P10-150 mm
	6,7,8	2P10-85 mm	2P10-150 mm
	9,10,11	2P10-85 mm	2P10-150 mm
	12,13,14	2P10-85 mm	2P10-150 mm
	15	2P10-85 mm	2P10-150 mm

Finally, it can be concluded that the stirrup used in plastic joint area of beams BI1X and BI1Y in all stories is 2P10-85 mm, while outside of the plastic joint area is 2P10-150 mm.

5.6.2 Secondary Beam Reinforcement Design

Like the main beams, the support area of the secondary beams also starts from the edge of the beam until a quarter (1/4) of the beam length on both sides, while the middle part of the beam is called the middle span area.

The ultimate moments (Mu) of the secondary beams are obtained from the ETABS model. These ultimate moments are then checked to be redistributed. To check if the moment needs redistribution, the percentage of positive moments against negative moments in the support area is calculated. Meanwhile, for the middle span area, the percentage of negative moments against positive moments is calculated. If the percentage shows a value larger than or equal to 50%, the moment does not need to be redistributed. On the other hand, if the percentage is less than 50%, the moment needs to be redistributed. The moment redistribution checking is as follows.

Table 5.50 BA1X Moment Redistribution Checking

Beam	Story	Support				Middle Span			
		M+	M-	Moment (%)	Status	M+	M-	Moment (%)	Status
BA1X	1	140.28	-240.30	58.38	OK	139.96	-116.50	83.24	OK
	2,3,4,5	187.34	-305.57	61.31	OK	177.57	-155.65	87.66	OK
	6,7,8	181.40	-302.43	59.98	OK	173.03	-153.70	88.83	OK
	9,10,11	156.94	-275.06	57.05	OK	153.72	-137.31	89.32	OK
	12,13,14	115.68	-231.76	49.91	Redis	124.21	-105.48	84.92	OK
	15	71.35	-156.42	45.61	Redis	81.70	-65.66	80.37	OK

Table 5.51 BA1Y Moment Redistribution Checking

Beam	Story	Support				Middle Span			
		M+	M-	Moment (%)	Status	M+	M-	Moment (%)	Status
BA1Y	1	55.03	-159.87	34.42	Redis	142.51	-36.31	25.48	Redis
	2,3,4,5	75.92	-210.66	36.04	Redis	176.41	-66.73	37.83	Redis
	6,7,8	72.43	-206.33	35.10	Redis	172.42	-64.22	37.25	Redis
	9,10,11	59.81	-187.45	31.91	Redis	158.07	-50.53	31.97	Redis
	12,13,14	35.98	-152.72	23.56	Redis	132.18	-24.37	18.44	Redis
	15	39.87	-107.41	37.12	Redis	94.08	-3.43	3.65	Redis

From the tables above, it is found that the moments of beams BA1X need redistribution in some of the upper stories, while the moments of beams BA1Y need redistribution in all of the stories. The moments that need redistribution are then analyzed as follows.

Table 5.52 Moment Redistribution Analysis of Secondary Beam BA1X

Support Area							
Story	% Redistribution	% Red. x M-	M-	ΣM	M+	Moment (%)	Status
1	0	0	240.30	761.16	140.28	58.38	OK
2,3,4,5	0	0	305.57	985.82	187.34	61.31	OK
6,7,8	0	0	302.43	967.67	181.40	59.98	OK
9,10,11	0	0	275.06	864.00	156.94	57.05	OK
12,13,14	1	2	229.44	694.87	117.99	51.43	OK
15	3	4.69	151.73	455.56	76.05	50.12	OK
Middle Span Area							
Story	% Redistribution	% Red. x M+	M-	ΣM	M+	Moment (%)	Status
1	0	0	139.96	512.92	116.50	83.24	OK
2,3,4,5	0	0	177.57	666.45	155.65	87.66	OK
6,7,8	0	0	173.03	653.45	153.70	88.83	OK
9,10,11	0	0	153.72	582.07	137.31	89.32	OK
12,13,14	0	0	124.21	459.38	105.48	84.92	OK
15	0	0	81.70	294.73	65.66	80.37	OK

Table 5.53 Moment Redistribution Analysis of Secondary Beam BA1Y

Support Area							
Story	% Redistribution	% Red. x M-	M-	ΣM	M+	Moment (%)	Status
1	11	17.59	142.28	429.80	72.62	51.04	OK
2,3,4,5	10	21.07	189.59	573.17	96.99	51.16	OK
6,7,8	10	20.63	185.69	557.51	93.06	50.12	OK
9,10,11	13	24.37	163.08	494.52	84.18	51.62	OK
12,13,14	18	27.49	125.23	377.41	63.47	50.68	OK
15	9	9.67	97.74	294.57	49.54	50.68	OK
Middle Span Area							
Story	% Redistribution	% Red. x M+	M-	ΣM	M+	Moment (%)	Status
1	17	24.23	118.28	357.64	60.54	51.18	OK
2,3,4,5	9	15.88	160.53	486.28	82.61	51.46	OK
6,7,8	9	15.52	156.90	473.29	79.74	50.82	OK
9,10,11	13	20.55	137.52	417.20	71.08	51.69	OK
12,13,14	22	29.08	103.10	313.11	53.45	51.84	OK
15	31	29.16	64.91	195.02	32.60	50.22	OK

From the tables above, it can be concluded that the moments are safe after being redistributed. Hence, the final redistributed ultimate moments are as follows.

Table 5.54 Final Redistributed Ultimate Moments of Secondary Beam BA1X

BA1X	Support		Middle Span	
Story	M+	M-	M+	M-
1	140.28	240.30	139.96	116.50
2,3,4,5	187.34	305.57	177.57	155.65
6,7,8	181.40	302.43	173.03	153.70
9,10,11	156.94	275.06	153.72	137.31
12,13,14	117.99	229.44	124.21	105.48
15	76.05	151.73	81.70	65.66

Table 5.55 Final Redistributed Ultimate Moments of Secondary Beam BA1Y

BA1Y	Support		Middle Span	
Story	M+	M-	M+	M-
1	72.62	142.28	118.28	60.54
2,3,4,5	96.99	189.59	160.53	82.61
6,7,8	93.06	185.69	156.90	79.74
9,10,11	84.18	163.08	137.52	71.08
12,13,14	63.47	125.23	103.10	53.45
15	49.54	97.74	64.91	32.60

The next step is to design the reinforcement of the secondary beams BA1X and BA1Y in each story. The reinforcement design example of the secondary beam BA1X support area in the first story is as follows.

Mu-	= 240.30 kN-m
Mu+	= 140.28 kN-m

The material properties are as follows.

φ	= 0.9
f'c	= 35 MPa
Ɛc	= 0.003
β	= 0.80
fy	= 400 MPa
E	= 200000 MPa
Ɛy	= 0.002

Meanwhile, the dimensions and details of the secondary beam BA1X support area are as follows.

Dp (flexural)	= 25 mm
Ds (shear)	= 10 mm
H	= 700 mm
B	= 350 mm
Concrete cover	= 40 mm
Reinforcement spacing (s)	= 25 mm
ds = ds'	= 62.5 mm
d = d'	= 637.5 mm

The assumption of the number of reinforcements needed is analyzed as follows.

1. Tensile moment

$$Mn_1 = \varphi \cdot Mu^{-} \cdot R = 0.9 \cdot 240.30 \cdot 0.5$$

$$Mn_1 = 108.13 \text{ kN-m} = 108133650 \text{ N-mm}$$

2. Quadratic formula to determine a value

Using the following quadratic formula to determine a value:

$$Mn = 0.85 \cdot f'c \cdot a \cdot b \cdot \left(d - \frac{a}{2}\right)$$

$$108133650 = 0.85 \cdot 35 \cdot a \cdot 350 \cdot \left(637.5 - \frac{a}{2}\right)$$

$$108133650 = 6637968.75a - 5206.25a^2$$

$$5206.25a^2 - 6637968.75a + 108133650 = 0$$

From the quadratic formula, the value of a is obtained 16.50 mm.

3. Compressive and tensile area

$$As_1 = \frac{0.85 \cdot f'c \cdot a \cdot b}{fy} = \frac{0.85 \cdot 35 \cdot 16.50 \cdot 350}{400} = 429.61 \text{ mm}^2$$

$$Mn_2 = Mu^- - Mn_1 = 240.30 - 108.13$$

$$Mn_2 = 132.16 \text{ kN-m} = 132163350 \text{ N-mm}$$

$$Ts_2 = \frac{Mn_2}{fy} = \frac{132163350}{400} = 330408.38 \text{ N}$$

$$As_2 = \frac{Ts_2}{fy} = \frac{330408.38}{400} = 826.02 \text{ mm}^2$$

$$As_{tensile} = As_1 + As_2 = 429.61 + 826.02 = 1255.64 \text{ mm}^2$$

$$As_{compression} = As_2 = 826.02 \text{ mm}^2$$

$$As_{1D} = \frac{1}{4}\pi \cdot Dp^2 = \frac{1}{4}\pi \cdot 25^2 = 490.87 \text{ mm}^2$$

4. Number of reinforcements (n)

$$n_{upper} = \frac{As_{tensile}}{As_{1D}} = \frac{1255.64}{490.87} = 2.56 \approx 3$$

$$n_{lower} = \frac{As_{compression}}{As_{1D}} = \frac{826.02}{490.87} = 1.68 \approx 2$$

Check spacing:

$$s = \frac{B-(2 \cdot \text{Concrete cover})-(2 \cdot Ds)-(n \cdot Dp)}{n-1} = 87.5 \text{ mm} > 25 \text{ mm (OK)}$$

Hence, the number of reinforcements for secondary beam BA1X in support area is obtained 3 in the upper (tensile) area and 2 in the lower (compression) area.

The analysis is then continued to determine the available or nominal moments. The analysis of the negative nominal moment of secondary beam BA1X in support area is as follows.

1. Area and reinforcement

$$As^- = n_{upper} \cdot As_{1D} = 3 \cdot 490.87 \text{ mm}^2 = 1472.62 \text{ mm}^2$$

$$As^+ = n_{lower} \cdot As_{1D} = 2 \cdot 490.87 \text{ mm}^2 = 981.75 \text{ mm}^2$$

$$ds = ds' = 62.5 \text{ mm}$$

$$d = d' = 637.5 \text{ mm}$$

2. Reinforcement condition assumption

Tensile area = yielded

Compression area = not yet yielded

3. Quadratic formula to determine c value

Using the following quadratic formula to determine c value:

$\mathrm{As}^{-} \cdot \mathrm{fy} = 0.85 \cdot \mathrm{f'c} \cdot \beta \cdot \mathrm{b} + \left(\frac{\mathrm{c}-\mathrm{ds}'}{\mathrm{c}}\right) \cdot \varepsilon\mathrm{c} \cdot \mathrm{Es} \cdot \mathrm{As}^{+}$

$589048.62\mathrm{c} = 8330\mathrm{c}^2 - 36815538.91 + 589048.62\mathrm{c}$

$8330\mathrm{c}^2 - 36815538.91 = 0$

From the quadratic formula, the value of c is obtained 66.48 mm.

Meanwhile, the value of a is calculated as follows.

$\mathrm{a} = \mathrm{c} \cdot \beta = 66.48 \cdot 0.80 = 53.18$ mm

4. Value of fs

$\mathrm{fs} = \left(\frac{\mathrm{c}-\mathrm{d}\ \ '}{\mathrm{c}}\right) \cdot \varepsilon\mathrm{c} \cdot \mathrm{Es} = \left(\frac{66.48-6\ \ .5}{66.48}\right) \cdot 0.003 \cdot 200000 = 35.92$ MPa

5. Negative moment (M^{-})

$\mathrm{Mn}_{\mathrm{cc}} = 0.85 \cdot \mathrm{f'c} \cdot \mathrm{a} \cdot \mathrm{b} \cdot \left(\mathrm{d} - \frac{\mathrm{a}}{2}\right)$

$\mathrm{Mn}_{\mathrm{cc}} = 0.85 \cdot 35 \cdot 53.18 \cdot 350 \cdot \left(637.5 - \frac{53.18}{2}\right) = 338309196.77$ N-mm

$\mathrm{Mn}_{\mathrm{cs}} = \mathrm{As}^{+} \cdot \mathrm{fs} \cdot (\mathrm{d} - \mathrm{ds}') = 981.75 \cdot 35.92 \cdot (637.5 - 62.5)$

$\mathrm{Mn}_{\mathrm{cs}} = 20278860.56$ N-mm

$\mathrm{Mn} = \mathrm{Mn}_{\mathrm{cc}} + \mathrm{Mn}_{\mathrm{cs}} = 338309196.77 + 20278860.56$

$\mathrm{Mn} = 358588057.34$ N-mm $= 358.59$ kN-m

$\varphi\mathrm{Mn} = 0.9 \cdot 358.59 = 322.73$ kN-m

Check towards Mu:

$\varphi\mathrm{Mn} > \mathrm{Mu}^{-}$

$322.73 > 240.30$ (SAFE)

Meanwhile, the analysis of the positive nominal moment of secondary beam BA1X in support area is as follows.

1. Area and reinforcement

$\mathrm{As}^{+} = \mathrm{n}_{\mathrm{lower}} \cdot \mathrm{As}_{1\mathrm{D}} = 2 \cdot 490.87\ \mathrm{mm}^2 = 981.75\ \mathrm{mm}^2$

$\mathrm{As}^{-} = \mathrm{n}_{\mathrm{upper}} \cdot \mathrm{As}_{1\mathrm{D}} = 3 \cdot 490.87\ \mathrm{mm}^2 = 1472.62\ \mathrm{mm}^2$

$\mathrm{ds} = \mathrm{ds}' = 62.5$ mm

$\mathrm{d} = \mathrm{d}' = 637.5$ mm

2. Reinforcement condition assumption

Tensile area = yielded

Compression area = not yet yielded

3. Quadratic formula to determine c value

Using the following quadratic formula to determine c value:

$$As^{+} \cdot fy = 0.85 \cdot f'c \cdot \beta \cdot b + \left(\frac{c-ds}{c}\right) \cdot \varepsilon c \cdot Es \cdot As^{-}$$

$$392699.08c = 8330c^2 - 55223308.36 + 883572.93c$$

$$8330c^2 + 490873.85c - 55223308.36 = 0$$

From the quadratic formula, the value of c is obtained 57.12 mm.

Meanwhile, the value of a is calculated as follows.

$$a = c \cdot \beta = 57.12 \cdot 0.80 = 45.70 \text{ mm}$$

4. Value of fs

$$fs = \left(\frac{c-ds}{c}\right) \cdot \varepsilon c \cdot Es = \left(\frac{57.12-62.5}{57.12}\right) \cdot 0.003 \cdot 200000 = -56.46 \text{ MPa}$$

5. Positive moment (M^{+})

$$Mn_{cc} = 0.85 \cdot f'c \cdot a \cdot b \cdot \left(d' - \frac{a}{2}\right)$$

$$Mn_{cc} = 0.85 \cdot 35 \cdot 45.70 \cdot 350 \cdot \left(637.5 - \frac{45.70}{2}\right) = 292479021.36 \text{ N-mm}$$

$$Mn_{cs} = As^{-} \cdot fs \cdot (d' - ds) = 1472.62 \cdot -56.46 \cdot (637.5 - 62.5)$$

$$Mn_{cs} = -47809631.05 \text{ N-mm}$$

$$Mn = Mn_{cc} + Mn_{cs} = 292479021.36 - 47809631.05$$

$$Mn = 244669390.31 \text{ N-mm} = 244.67 \text{ kN-m}$$

$$\varphi Mn = 0.9 \cdot 244.67 = 220.20 \text{ kN-m}$$

Check towards Mu:

$$\varphi Mn > Mu^{+}$$

$$220.20 > 140.28 \text{ (SAFE)}$$

The analysis is then continued to determine the probable moments (Mpr). The analysis of the negative probable moment of secondary beam BA1X in support area is as follows.

1. Area and reinforcement

$$As_{tensile} = n_{upper} \cdot As_{1D} = 3 \cdot 490.87 \text{ mm}^2 = 1472.62 \text{ mm}^2$$

$As_{compression} = n_{lower} \cdot As_{1D} = 2 \cdot 490.87 \text{ mm}^2 = 981.75 \text{ mm}^2$

$ds = ds' = 62.5 \text{ mm}$

$d = d' = 637.5 \text{ mm}$

$ø_{os} = 1.25$

2. Reinforcement condition assumption

   Tensile area = yielded

   Compression area = not yet yielded

3. Quadratic formula to determine c value

   Using the following quadratic formula to determine c value:

   $ø_{os} \cdot As_{tensile} \cdot fy = 0.85 \cdot f'c \cdot \beta \cdot b + \left(\frac{c-ds'}{c}\right) \cdot \varepsilon c \cdot Es \cdot As_{compression}$

   $736310.78c = 8330c^2 - 36815538.91 + 589048.62c$

   $8330c^2 - 147262.16c - 36815538.91 = 0$

   From the quadratic formula, the value of c is obtained 75.90 mm.

   Meanwhile, the value of a is calculated as follows.

   $a = c \cdot \beta = 75.90 \cdot 0.80 = 60.72 \text{ mm}$

4. Value of fs

   $fs = \left(\frac{c-ds'}{c}\right) \cdot \varepsilon c \cdot Es = \left(\frac{75.90-62.5}{75.90}\right) \cdot 0.003 \cdot 200000 = 105.96 \text{ MPa}$

5. Check Ts = Cc + Cs

   $Cc = 0.85 \cdot f'c \cdot a \cdot b = 0.85 \cdot 35 \cdot 60.72 \cdot 350 = 632285.69 \text{ N}$

   $Cs = As_{compression} \cdot fs = 981.75 \cdot 105.96 = 104025.09 \text{ N}$

   $Ts = ø_{os} \cdot As_{tensile} \cdot fy = 1.25 \cdot 1472.62 \cdot 400 = 736310.78 \text{ N}$

   Check Ts = Cc + Cs:

   $Ts = Cc + Cs$

   $736310.78 = 632285.69 + 104025.09$

   $736310.78 = 736310.78$ (OK)

6. Tensile probable moment (Mpr^-)

   $Mn_{cc} = 0.85 \cdot f'c \cdot a \cdot b \cdot \left(d - \frac{a}{2}\right)$

   $Mn_{cc} = 0.85 \cdot 35 \cdot 60.72 \cdot 350 \cdot \left(837.5 - \frac{60.72}{2}\right) = 383884758.62 \text{ N-mm}$

$\mathrm{Mn_{cs}} = \mathrm{As_{compression}} \cdot \mathrm{fs} \cdot (\mathrm{d} - \mathrm{ds}') = 981.75 \cdot 105.96 \cdot (837.5 - 62.5)$

$\mathrm{Mn_{cs}} = 59814426.21$ N-mm

$\mathrm{Mn} = \mathrm{Mn_{cc}} + \mathrm{Mn_{cs}} = 383884758.62 + 59814426.21$

$\mathrm{Mn} = 443699184.83$ N-mm $= 443.70$ kN-m

$\varphi\mathrm{Mn} = 0.9 \cdot 443.70 = 399.33$ kN-m

Meanwhile, the analysis of the positive probable moment of secondary beam BA1X in support area is as follows.

1. Area and reinforcement

   $\mathrm{As_{compression}} = \mathrm{n_{upper}} \cdot \mathrm{As_{1D}} = 2 \cdot 490.87\ \mathrm{mm^2} = 981.75\ \mathrm{mm^2}$

   $\mathrm{As_{tensile}} = \mathrm{n_{lower}} \cdot \mathrm{As_{1D}} = 3 \cdot 490.87\ \mathrm{mm^2} = 1472.62\ \mathrm{mm^2}$

   $\mathrm{ds} = \mathrm{ds}' = 62.5\ \mathrm{mm}$

   $\mathrm{d} = \mathrm{d}' = 637.5\ \mathrm{mm}$

   $\text{ø}_{\mathrm{os}} = 1.25$

2. Reinforcement condition assumption

   Tensile area = yielded

   Compression area = not yet yielded

3. Quadratic formula to determine c value

   Using the following quadratic formula to determine c value:

   $\text{ø}_{\mathrm{os}} \cdot \mathrm{As_{tensile}} \cdot \mathrm{fy} = 0.85 \cdot \mathrm{f'c} \cdot \beta \cdot \mathrm{b} + \left(\frac{\mathrm{c-ds}}{\mathrm{c}}\right) \cdot \varepsilon\mathrm{c} \cdot \mathrm{Es} \cdot \mathrm{As_{compression}}$

   $490873.85\mathrm{c} = 8330\mathrm{c}^2 - 55223308.36 + 883572.93\mathrm{c}$

   $8330\mathrm{c}^2 + 392699.08\mathrm{c} - 55223308.36 = 0$

   From the quadratic formula, the value of c is obtained 61.19 mm.

   Meanwhile, the value of a is calculated as follows.

   $\mathrm{a} = \mathrm{c} \cdot \beta = 61.19 \cdot 0.80 = 48.95\ \mathrm{mm}$

4. Value of fs

   $\mathrm{fs} = \left(\frac{\mathrm{c-ds}}{\mathrm{c}}\right) \cdot \varepsilon\mathrm{c} \cdot \mathrm{Es} = \left(\frac{61.19-6\ \ .5}{61.19}\right) \cdot 0.003 \cdot 200000 = -12.81\ \mathrm{MPa}$

5. Check Ts = Cc + Cs

   $\mathrm{Cc} = 0.85 \cdot \mathrm{f'c} \cdot \mathrm{a} \cdot \mathrm{b} = 0.85 \cdot 35 \cdot 48.95 \cdot 350 = 509740.61$ N

   $\mathrm{Cs} = \mathrm{As_{tensile}} \cdot \mathrm{fs} = 1472.62 \cdot -12.81 = -18866.76$ N

   $\mathrm{Ts} = \text{ø}_{\mathrm{os}} \cdot \mathrm{As_{compression}} \cdot \mathrm{fy} = 1.25 \cdot 981.75 \cdot 400 = 490873.85$ N

Check Ts = Cc + Cs:

$\mathrm{Ts} = \mathrm{Cc} + \mathrm{Cs}$

$490873.85 = 509740.61 - 18866.76$

$490873.85 = 490873.85$ (OK)

6. Compression probable moment (Mpr^{+})

$\mathrm{Mn_{cc}} = 0.85 \cdot \mathrm{f'c} \cdot \mathrm{a} \cdot \mathrm{b} \cdot \left(\mathrm{d} - \frac{\mathrm{a}}{2}\right)$

$\mathrm{Mn_{cc}} = 0.85 \cdot 35 \cdot 48.95 \cdot 350 \cdot \left(837.5 - \frac{48.95}{2}\right) = 312482544.43$ N-mm

$\mathrm{Mn_{cs}} = \mathrm{As_{tensile}} \cdot \mathrm{fs} \cdot (\mathrm{d} - \mathrm{ds'}) = 1472.62 \cdot -12.81 \cdot (837.5 - 62.5)$

$\mathrm{Mn_{cs}} = -10848385.67$ N-mm

$\mathrm{Mn} = \mathrm{Mn_{cc}} + \mathrm{Mn_{cs}} = 312482544.43 - 10848385.67$

$\mathrm{Mn} = 301634158.76$ N-mm $= 301.63$ kN-m

$\varphi\mathrm{Mn} = 0.9 \cdot 301.63 = 271.47$ kN-m

Finally, all the results of the number of reinforcement (n), nominal and probable moments of secondary beam BA1X in support area are recapitulated in the following table.

Table 5.56 Moment and Flexural Reinforcement Results of BA1X Support Area

BA1X Support Area		
Mu-	240.30	kNm
Mu+	140.28	kNm
M-	322.73	kNm
M+	220.20	kNm
Mpr-	399.33	kNm
Mpr+	271.47	kNm
Upper Reinforcement	3	piece
	3D25	
Lower Reinforcement	2	piece
	2D25	

The number of reinforcement (n) and moment analysis are then conducted for the middle span with the same steps. The difference between support and middle span area is as follows.

1. Support area: Upper reinforcement area is the negative or tensile area, and lower reinforcement area is the positive or compression area.

2. Middle span area: Upper reinforcement area is the positive or compression area, and lower reinforcement area is the negative or tensile area.

The results of the number of reinforcement (n), nominal and probable moments of secondary beam BA1X in middle span area are recapitulated in the following table.

Table 5.57 Moment and Flexural Reinforcement Results of BA1X Middle Span Area

BA1X Middle Span Area		
Mu-	116.50	kNm
Mu+	139.96	kNm
M-	220.02	kNm
M+	205.19	kNm
Mpr-	271.44	kNm
Mpr+	271.44	kNm
Upper Reinforcement	2	piece
	2D25	
Lower Reinforcement	2	piece
	2D25	

This analysis is conducted on both secondary beams BA1X and BA1Y in all the story groups. The results are as follows.

Table 5.58 Recapitulation of Moment and Flexural Reinforcement Results of BA1X

Beam	Description	Story												Unit
		1		2,3,4,5		6,7,8		9,10,11		12,13,14		15		
		Support	Middle Span	Support	Middle Span	Support	Middle Span	Support	Middle Span	Support	Middle Span	Support	Middle Span	
BA1X	Mu-	240.30	116.50	305.57	155.65	302.43	153.70	229.44	105.48	151.73	65.66	237.84	53.65	kNm
	Mu+	140.28	139.96	187.34	177.57	181.40	173.03	117.99	124.21	76.05	81.70	129.67	106.92	kNm
	M-	322.73	220.02	433.04	294.43	433.04	294.43	432.93	294.43	294.43	294.43	432.93	294.43	kNm
	M+	220.20	205.19	433.04	271.32	433.04	271.32	294.90	271.32	294.43	271.32	294.90	271.32	kNm
	Mpr-	399.33	271.44	536.47	363.71	536.47	363.71	536.53	363.71	363.71	363.71	536.53	363.71	kNm
	Mpr+	271.47	271.44	536.47	363.71	536.47	363.71	363.99	363.71	363.71	363.71	363.99	363.71	kNm
	Upper reinforcement	3	2	3	2	3	2	3	2	2	2	3	2	piece
		3D25	2D25	3D25	2D25	3D25	2D25	3D25	2D25	2D25	2D25	3D25	2D25	
	Lower reinforcement	2	2	3	2	3	2	2	2	2	2	2	2	piece
		2D25	2D25	3D25	2D25	3D25	2D25	2D25	2D25	2D25	2D25	2D25	2D25	

Table 5.59 Recapitulation of Moment and Flexural Reinforcement Results of BA1Y

Beam	Description	Story												Unit
		1		2,3,4,5		6,7,8		9,10,11		12,13,14		15		
		Support	Middle Span	Support	Middle Span	Support	Middle Span	Support	Middle Span	Support	Middle Span	Support	Middle Span	
BA1Y	Mu-	142.28	60.54	189.59	82.61	185.69	79.74	163.08	71.08	125.23	53.45	97.74	32.60	kNm
	Mu+	72.62	118.28	96.99	160.53	93.06	156.90	84.18	137.52	63.47	103.10	49.54	64.91	kNm
	M-	145.17	145.17	294.43	294.43	294.43	294.43	294.43	294.43	294.43	294.43	294.43	294.43	kNm
	M+	145.17	137.25	294.43	271.32	294.43	271.32	294.43	271.32	294.43	271.32	294.43	271.32	kNm
	Mpr-	178.63	178.63	363.71	363.71	363.71	363.71	363.71	363.71	363.71	363.71	363.71	363.71	kNm
	Mpr+	178.63	178.63	363.71	363.71	363.71	363.71	363.71	363.71	363.71	363.71	363.71	363.71	kNm
	Upper reinforcement	2	2	2	2	2	2	2	2	2	2	2	2	piece
		2D25	2D25	2D25	2D25	2D25	2D25	2D25	2D25	2D25	2D25	2D25	2D25	
	Lower reinforcement	2	2	2	2	2	2	2	2	2	2	2	2	piece
		2D25	2D25	2D25	2D25	2D25	2D25	2D25	2D25	2D25	2D25	2D25	2D25	

For the shear reinforcement design of secondary beams, the shear force values obtained from the analysis of the ETABS model due to gravitational load (Vg) within and outside the plastic joint area (Lo) can be seen in the following table.

Table 5.60 ETABS Shear Force Values for Shear Reinforcement Design of Secondary Beams

Beam	Story	PJ	≠PJ	PJ
		Vg left	Vg upper/lower	Vg right
BA1X	1	-127.20	99.86	127.20
	2,3,4,5	-117.29	92.10	117.29
	6,7,8	-117.23	91.98	117.23
	9,10,11	-117.20	91.91	117.20
	12,13,14	-117.19	91.87	117.19
	15	-78.44	58.20	78.44
BA1Y	1	-73.72	69.04	73.72
	2,3,4,5	-96.96	64.59	96.96
	6,7,8	-98.62	61.81	98.62
	9,10,11	-99.68	60.29	99.68
	12,13,14	-100.18	59.36	100.18
	15	-58.57	52.46	58.57

The initial data of main beam BA1X material properties for shear reinforcement design in the first story is as follows.

L_{beam} = 8 m = 8000 mm

$B_{left\ beam}$ = 450 mm

$B_{right\ beam}$ = 450 mm

L_{netto} = 8000 – 0.5(450) – 0.5(450) = 7550 mm = 7.55 m

Mpr^{-} = 271.44 kNm

Mpr^+ = 271.44 kNm

B_{beam} = 350 mm

H_{beam} = 700 mm

f'c = 35 MPa

$fy_{stirrup}$ = 360 MPa

D_{shear} = 10 mm

φ = 0.75

H^- = 700 mm

$D_{flexural}$ = 25 mm

The shear force analysis to determine the shear reinforcement (stirrup) of the main beam BI1X in the first story as an example is as follows.

1. Shear force due to gravitational load (Vg)

   Vg_{left} = 127.20 kN → Vg_{left}/φ = 127.20/0.75 = 169.60 kN

   Vg_{right} = 127.20 kN → Vg_{left}/φ = -(127.20/0.75) = -169.60 kN

   Vg_{upper} = 99.86 kN → Vg_{upper}/φ = 99.86/0.75 = 133.14 kN

   Vg_{lower} = 99.86 kN → Vg_{lower}/φ = -(99.86/0.75) = -133.14 kN

   The shear force diagram (SFD) obtained from the Vg values is as follows.

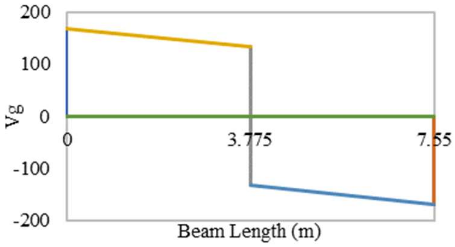


Figure 5.15 SFD of Vg in BA1X Story 1

2. Shear force due to earthquake load (Ve)

   Earthquake direction is taken from the left:

$$Ve_{left} = -\left(\frac{Mpr^+}{L_{netto} \cdot \varphi} + \frac{Mpr^-}{L_{netto} \cdot \varphi}\right) = -\left(\frac{271.44}{7.55 \cdot 0.75} + \frac{271.44}{7.55 \cdot 0.75}\right) = -95.87 \text{ kN}$$

$$Ve_{right} = \left(\frac{Mpr^-}{L_{netto} \cdot \varphi} + \frac{Mpr^+}{L_{netto} \cdot \varphi}\right) = \left(\frac{271.44}{7.55 \cdot 0.75} + \frac{271.44}{7.55 \cdot 0.75}\right) = 95.87 \text{ kN}$$

   Because the earthquake direction is from the left, the Ve is taken -95.87 kN. Meanwhile, the shear force diagram (SFD) obtained from the Ve values is as follows.

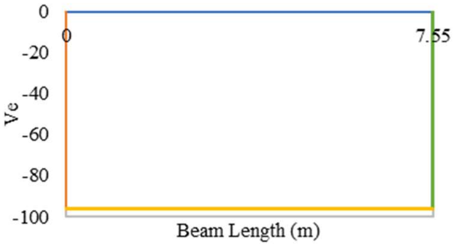


Figure 5.16 SFD of Ve in BA1X Story 1

3. Ultimate shear force (Vu) combination of Vg and Ve

$Vu_{left} = Vg_{left} + Ve = 127.20 + |-95.87| = 265.48$ kN

$Vu_{right} = Vg_{right} + Ve = (-127.20) + (-95.87) = -265.48$ kN

$Vu_{upper} = Vg_{upper} + Ve = 99.86 + |-95.87| = 229.01$ kN

$Vu_{lower} = Vg_{lower} + Ve = (-99.86) + (-95.87) = -229.01$ kN

The shear force diagram (SFD) obtained from the Vu combination values is as follows.

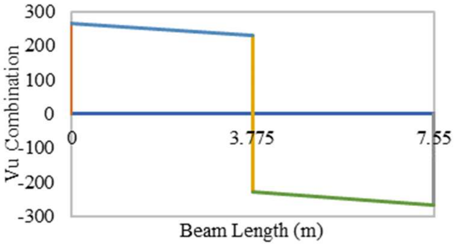


Figure 5.17 SFD of Vu Combination in BA1X Story 1

4. Diagram dimension

The dimensions of the shear force diagrams are determined as follows.

$$c = \frac{Vu_{left} - V\ \ upper}{(Vu_{left} - Vu_{upper}) + (Vu_{right} - Vu_{lower})} \cdot L_{netto}$$

$$c = \frac{265.48 - 229.01}{(265.48 - 229.01) + |(-265.48) - (-229.01)|} \cdot 7550 = 3775 \text{ mm} = 3.775 \text{ m}$$

$$d = L_{netto} - c = 7550 - 3775 = 3775 \text{ mm} = 3.775 \text{ m}$$

x is the bigger value between c and d, so the value is concluded as c = 3775 mm.

5. Plastic joint area

$$Vc = \frac{1}{6} \cdot \sqrt{f'c} \cdot B_{beam} \cdot H^{-} = \frac{1}{6} \cdot \sqrt{35} \cdot 350 \cdot 700$$

$$Vc = 241573.26 \text{ N} = 241.57 \text{ kN}$$

If the value of Ve > Vg_{right}, the Vs_1 value is determined as the bigger value between Vu_{left} and Vu_{right}. Meanwhile, if it is the other way around, the Vs_1 value is subtracted by the value of Vc.

$\mathrm{Ve} < \mathrm{Vg_{right}}$

$95.87\ \mathrm{kN} < 127.20\ \mathrm{kN}$

$\mathrm{Vs_1} = \mathrm{Vu_{right}} - \mathrm{Vc} = 265.48 - 241.57 = 23.90\ \mathrm{kN}$

$\mathrm{Av} = \frac{1}{4} \cdot \pi \cdot \mathrm{D_{shear}}^2 = \frac{1}{4} \cdot \pi \cdot 10^2 = 78.54\ \mathrm{mm^2}$

$\mathrm{n_{stirrup}} = 2$

$\mathrm{fy_{stirrup}} = 360\ \mathrm{N/mm^2} = 0.36\ \mathrm{kN/mm^2}$

$\mathrm{s} = \mathrm{n_{stirrup}} \cdot \frac{\mathrm{Av} \cdot \mathrm{fy_{stirrup}} \cdot \mathrm{H^-}}{\mathrm{Vs_1}} = 2 \cdot \frac{78.54 \cdot 0.36 \cdot 700}{23.90} = 1656.10\ \mathrm{mm}$

$\mathrm{s_{used}} = 85\ \mathrm{mm}$

Check:

a. $\frac{\mathrm{H^-}}{4} = \frac{700}{4} = 175\ \mathrm{mm}$

b. $8 \cdot \mathrm{D_{flexural}} = 8 \cdot 25 = 200\ \mathrm{mm}$

c. $24 \cdot \mathrm{D_{shear}} = 24 \cdot 10 = 240\ \mathrm{mm}$

d. $300\ \mathrm{mm}$

The minimum value of the requirements above is 175 mm.

$50\ \mathrm{mm} \leq \mathrm{s_{used}} \leq 175\ \mathrm{mm}$

$50\ \mathrm{mm} < 85\ \mathrm{mm} < 175\ \mathrm{mm}$ (OK)

Hence, the shear reinforcement (stirrup) used in the plastic joint area is 2P10-85 mm.

6. Outside plastic joint area

$\mathrm{V_{ats}} = \mathrm{Vu_{right}} - \mathrm{Ve} - \mathrm{Vg_{upper}} = 265.48 - 95.87 - 133.14 = 36.46\ \mathrm{kN}$

$\mathrm{x} = \frac{\mathrm{V_{ats}} \cdot (\mathrm{x} - \mathrm{H_{beam}})}{\mathrm{x}} = \frac{36.46 \cdot (3775.04 - 700)}{3775.04} = 22.94\ \mathrm{mm}$

$\mathrm{y} = \mathrm{x} + \mathrm{Vg_{upper}} + \mathrm{Ve} = 22.94 + 133.14 + 95.87 = 251.95\ \mathrm{kN}$

$\mathrm{Vs_2} = \mathrm{y} - \mathrm{Vc} = 251.95 - 241.57 = 10.38\ \mathrm{kN}$

$\mathrm{n_{stirrup}} = 2$

$\mathrm{fy_{stirrup}} = 360\ \mathrm{N/mm^2} = 0.36\ \mathrm{kN/mm^2}$

$$s = n_{stirrup} \cdot \frac{Av \cdot fy_{stirrup} \cdot H^{-}}{Vs_2} = 2 \cdot \frac{78.54 \cdot 0.36 \cdot 700}{10.38} = 3813.89 \text{ mm}$$

$$s_{used} = 150 \text{ mm}$$

Check:

$$50 \text{ mm} \le s_{used} \le \frac{H^{-}}{2}$$

$$50 \text{ mm} \le 150 \text{ mm} \le \frac{700}{2}$$

$$50 \text{ mm} < 150 \text{ mm} < 350 \text{ mm}$$ (OK)

Hence, the shear reinforcement (stirrup) used outside the plastic joint area is 2P10-150 mm.

Furthermore, the shear reinforcement design is conducted on both secondary beams BA1X and BA1Y in all the story groups. The results are as follows.

Table 5.61 Recapitulation Shear Reinforcement Results of BA1X

Beam	Story	Stirrup	
		Plastic Joint	Outside Plastic Joint
BA1X	1	2P10-85 mm	2P10-150 mm
	2,3,4,5	2P10-85 mm	2P10-150 mm
	6,7,8	2P10-85 mm	2P10-150 mm
	9,10,11	2P10-85 mm	2P10-150 mm
	12,13,14	2P10-85 mm	2P10-150 mm
	15	2P10-85 mm	2P10-150 mm

Table 5.62 Recapitulation Shear Reinforcement Results of BA1Y

Beam	Story	Stirrup	
		Plastic Joint	Outside Plastic Joint
BA1Y	1	2P10-85 mm	2P10-150 mm
	2,3,4,5	2P10-85 mm	2P10-150 mm
	6,7,8	2P10-85 mm	2P10-150 mm
	9,10,11	2P10-85 mm	2P10-150 mm
	12,13,14	2P10-85 mm	2P10-150 mm
	15	2P10-85 mm	2P10-150 mm

Finally, it can be concluded that the stirrup used in plastic joint area of beams BA1X and BA1Y in all stories is 2P10-85 mm, while outside of the plastic joint area is 2P10-150 mm.

5.6.3 Column Reinforcement Design

As previously designed, one type of column is used for the building, with dimensions of Ht×B equal to 1200×1000 mm. The flexural reinforcement design of the columns is as follows.

$\mathrm{Ag} = \mathrm{Ht} \times \mathrm{B} = 1200 \times 1000 = 1200000\ \mathrm{mm}^2$

$\mathrm{D}_{\mathrm{flexural}} = 32\ \mathrm{mm} \ \rightarrow\ \mathrm{As}_{\mathrm{flexural}} = \frac{1}{4} \cdot \pi \cdot \mathrm{D}_{\mathrm{flexural}}{}^{2} = \frac{1}{4} \cdot \pi \cdot 32^2 = 804.25\ \mathrm{mm}^2$

$\mathrm{D}_{\mathrm{shear}} = 10\ \mathrm{mm} \ \rightarrow\ \mathrm{As}_{\mathrm{shear}} = \frac{1}{4} \cdot \pi \cdot \mathrm{D}_{\mathrm{shear}}{}^{2} = \frac{1}{4} \cdot \pi \cdot 10^2 = 78.54\ \mathrm{mm}^2$

$\text{Concrete cover (Sb)} = 40\ \mathrm{mm}$

$\mathrm{d} = \mathrm{d}' = 40 + 10 + 0.5(32) = 66\ \mathrm{mm} = 6.6\ \mathrm{cm}$

$\mathrm{H} = \mathrm{H}' = \mathrm{Ht} - \mathrm{d}' = 1200 - 66 = 1134\ \mathrm{mm} = 113.4\ \mathrm{cm}$

$\mathrm{f}'\mathrm{c} = 35\ \mathrm{MPa} = 357\ \mathrm{kg/cm}^2$

$\beta = 0.85 - \frac{0.05(\mathrm{f}'\mathrm{c} - 28)}{7} = 0.85 - \frac{0.05(35-28)}{7} = 0.80$

$\mathrm{fy} = 400\ \mathrm{MPa} = 4080\ \mathrm{kg/cm}^2$

$\mathrm{Es} = 200000\ \mathrm{MPa} = 2038736\ \mathrm{kg/cm}^2$

$\varepsilon_{\mathrm{y}} = \frac{\mathrm{fy}}{\mathrm{Es}} = \frac{400}{200000} = 0.002$

$\varepsilon_{\mathrm{c}} = 0.003$

$\text{Ratio between Ag and reinforcement used} = 1.5\%$

$\mathrm{As}_{\mathrm{reinforcement}}\ (\mathrm{Ast}) = 1.5\% \times \mathrm{Ag} = 1.5\% \times 1200000 = 18000\ \mathrm{mm}^2$

The flexural reinforcement analysis is as follows.

$\mathrm{As}_{\mathrm{needed}} = \mathrm{Ast} = 18000\ \mathrm{mm}^2$

$\mathrm{As}_{\mathrm{flexural}} = 804.25\ \mathrm{mm}^2$

$\mathrm{n}_{\mathrm{needed}} = \frac{\mathrm{As}_{\mathrm{needed}}}{\mathrm{As}_{\mathrm{flexural}}} = \frac{18000}{804.25} = 22.38 \approx 24 \ \rightarrow\ 24\mathrm{D}32$

As the column does not have the same height (H) and width (B) dimension (not a perfect square), the reinforcement placement can be seen in the illustration shown in the following figure.

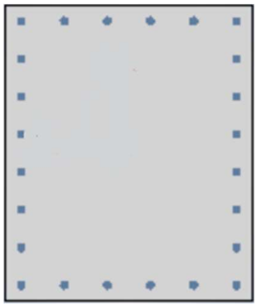


Figure 5.18 Illustration of Column Flexural Reinforcement

From the illustration, the number of reinforcements used in the X portal (with length as much as 1200 mm) has 8 reinforcements, while in the Y portal (with length as much as 1000 mm) has 6 reinforcements. It is required that the spacing between the reinforcements must not be less than the flexural reinforcement diameter, which is 32 mm. The spacing of the reinforcements is checked as follows.

$$n = 8 \ \rightarrow s = \frac{1200 \quad (2 \cdot Sb + 2 \cdot D_{shear} + n \cdot D_{flexural})}{n-1} = \frac{1200-(2 \cdot 40+2 \cdot 10+8 \cdot 32)}{8-1} = 120.6 \text{ mm}$$

$$n = 6 \ \rightarrow s = \frac{1000-(2 \cdot Sb + 2 \cdot D_{shear} + n \cdot D_{flexural})}{n-1} = \frac{1000 \quad (2 \cdot 40+2 \cdot 10+6 \cdot 32)}{6-1} = 141.6 \text{ mm}$$

As the spacings both surpass 32 mm, the spacings are considered to have fulfilled the requirement. Hence, the flexural reinforcement area for the X portal is as follows.

$$n = 8 \ \rightarrow \ As = As' = As_{flexural} \cdot n = 804.25 \cdot 8 = 6433.98 \text{ mm}^2 = 64.34 \text{ cm}^2$$

Meanwhile, the flexural reinforcement area for the Y portal is as follows.

$$n = 6 \ \rightarrow \ As = As' = As_{flexural} \cdot n = 804.25 \cdot 6 = 4825.49 \text{ mm}^2 = 48.25 \text{ cm}^2$$

To design the column flexural reinforcement as well as to obtain the nominal moments and axial loads of the column designed, the Mn-Pn diagram method is used. The analysis of Mn-Pn diagram according to the conditions is as follows.

The analysis example for column in the X portal in the first story is as follows.

1. Centric load

$$Cc = 0.85 \cdot f'c \cdot B \cdot Ht = 364100 \text{ kg}$$

$$Cs_1 = As(fy - 0.85 \cdot f'c) = 242982.54 \text{ kg}$$

$$Cs_2 = As'(fy - 0.85 \cdot f'c) = 242982.54 \text{ kg}$$

$$Pn = Cc + Cs_1 + Cs_2 = 4127.37 \text{ ton}$$

$$Mn = 0 \text{ ton-m}$$

2. Compression failure

$$\mathrm{Cb} = \frac{\varepsilon_c}{\varepsilon_c+\varepsilon_y} \cdot \mathrm{H} = \frac{0.003}{0.003+0.002} \cdot 113.4 = 68.02 \text{ cm}$$

Taking an example for n equal to 1.1, the calculation is as follows.

$$\mathrm{C} = \mathrm{n} \cdot \mathrm{Cb} = 74.83 \text{ cm}$$

$$\mathrm{a} = \beta \cdot \mathrm{C} = 59.86 \text{ cm}$$

$$\varepsilon_s' = \frac{\mathrm{C}-\mathrm{d}'}{\mathrm{C}} \cdot 0.003 = 0.00274$$

Check compression steel strain:

$$\varepsilon_s' > \varepsilon_y \ \rightarrow 0.00274 > 0.002 \text{ (Steel has yieldied)}$$

$$\varepsilon_s = \frac{\mathrm{h}-\mathrm{C}}{\mathrm{C}} \cdot 0.003 = 0.00155$$

Check tension steel strain:

$$\varepsilon_s > \varepsilon_y \ \rightarrow 0.00155 < 0.002 \text{ (Steel has not yieldied)}$$

$$\mathrm{Cc} = 0.85 \cdot \mathrm{f'c} \cdot \mathrm{a} \cdot \mathrm{B} = 1816462.46 \text{ kg}$$

$$\mathrm{Cs} = \mathrm{As}'(\mathrm{fy} - 0.85 \cdot \mathrm{f'c}) = 242982.54 \text{ kg}$$

$$\mathrm{Ts} = \mathrm{As} \cdot \mathrm{fs} = 202868.08 \text{ kg}$$

$$\mathrm{Pn} = \mathrm{Cc} + \mathrm{Cs} - \mathrm{Ts} = 1856.58 \text{ ton}$$

$$\mathrm{Mn} = \mathrm{Cc}\left(\frac{1}{2}\mathrm{Ht} - \frac{1}{2}\mathrm{a}\right) + \mathrm{Cs}\left(\frac{1}{2}\mathrm{Ht} - \mathrm{d}'\right) + \mathrm{Ts}\left(\frac{1}{2}\mathrm{Ht} - \mathrm{d}\right) = 784.29 \text{ ton-m}$$

This calculation process continues for several other values of n, which can be seen in the following table.

Table 5.63 Pn and Mn Values in Compression Failure Condition of the X Direction in the First Story

n.C	Result	a	Cc (kg)	Cs (kg)	Ts (kg)	Pn (ton)	Mn (ton-m)
1.009	68.64	54.91	1666191.48	242982.54	256654.92	1652.52	809.08
1.02	69.38	55.51	1684356.10	242982.54	249643.28	1677.70	806.21
1.08	73.46	58.77	1783435.87	242982.54	213912.22	1812.51	789.96
1.1	74.83	59.86	1816462.46	242982.54	202868.08	1856.58	784.29
1.2	81.63	65.30	1981595.41	242982.54	153169.43	2071.41	753.49
1.3	88.43	70.74	2146728.36	242982.54	111116.73	2278.59	717.78
1.4	95.23	76.19	2311861.31	242982.54	75071.55	2479.77	676.30
1.5	102.03	81.63	2476994.27	242982.54	43832.40	2676.14	628.40

3. Balance

$\mathrm{Cb} = \frac{\varepsilon_c}{\varepsilon_c+\varepsilon_y} \cdot \mathrm{H} = \frac{0.003}{0.003+0.002} \cdot 113.4 = 68.02\ \mathrm{cm}$

$\mathrm{ab} = \beta \cdot \mathrm{Cb} = 54.42\ \mathrm{cm}$

$\mathrm{Cc} = 0.85 \cdot \mathrm{f'c} \cdot \mathrm{ab} \cdot \mathrm{B} = 1651329.51\ \mathrm{kg}$

$\mathrm{Cs} = \mathrm{As'}(\mathrm{fy} - 0.85 \cdot \mathrm{f'c}) = 242982.54\ \mathrm{kg}$

$\mathrm{Ts} = \mathrm{As} \cdot \mathrm{fy} = 262506.46\ \mathrm{kg}$

$\mathrm{Pnb} = \mathrm{Cc} + \mathrm{Cs} - \mathrm{Ts} = 1631.81\ \mathrm{ton}$

$\mathrm{Mnb} = \mathrm{Cc}\left(\frac{1}{2}\mathrm{Ht} - \frac{1}{2}\mathrm{ab}\right) + \mathrm{Cs}\left(\frac{1}{2}\mathrm{Ht} - \mathrm{d'}\right) + \mathrm{Ts}\left(\frac{1}{2}\mathrm{Ht} - \mathrm{d}\right) = 811.41$ ton-m

4. Tension failure

$\mathrm{Cb} = \frac{\varepsilon_c}{\varepsilon_c+\varepsilon_y} \cdot \mathrm{H} = \frac{0.003}{0.003+0.002} \cdot 113.4 = 68.02\ \mathrm{cm}$

Taking an example for n equal to 0.9, the calculation is as follows.

$\mathrm{C} = 0.9 \cdot \mathrm{Cb} = 61.22\ \mathrm{cm}$

$\mathrm{a} = \beta \cdot \mathrm{C} = 48.98\ \mathrm{cm}$

$\varepsilon_s' = \frac{\mathrm{C}-\mathrm{d'}}{\mathrm{C}} \cdot 0.003 = 0.00268$

Check compression steel strain:

$\varepsilon_s' > \varepsilon_y \rightarrow 0.00268 > 0.002$ (Steel has yieldied)

$\varepsilon_s = \frac{\mathrm{h}-\mathrm{C}}{\mathrm{C}} \cdot 0.003 = 0.00256$

Check tension steel strain:

$\varepsilon_s > \varepsilon_y \rightarrow 0.00256 > 0.002$ (Steel has yieldied)

$\mathrm{Cc} = 0.85 \cdot \mathrm{f'c} \cdot \mathrm{a} \cdot \mathrm{B} = 1486196.56\ \mathrm{kg}$

$\mathrm{Cs} = \mathrm{As'}(\mathrm{fy} - 0.85 \cdot \mathrm{f'c}) = 242982.54\ \mathrm{kg}$

$\mathrm{Ts} = \mathrm{As} \cdot \mathrm{fy} = 262506.46\ \mathrm{kg}$

$\mathrm{Pn} = \mathrm{Cc} + \mathrm{Cs} - \mathrm{Ts} = 1466.67\ \mathrm{ton}$

$\mathrm{Mn} = \mathrm{Cc}\left(\frac{1}{2}\mathrm{Ht} - \frac{1}{2}\mathrm{a}\right) + \mathrm{Cs}\left(\frac{1}{2}\mathrm{Ht} - \mathrm{d'}\right) + \mathrm{Ts}\left(\frac{1}{2}\mathrm{Ht} - \mathrm{d}\right) = 797.70$ ton-m

This calculation process continues for several other values of n, which can be seen in the following table.

Table 5.64 Pn and Mn Values in Tension Failure Condition of the X Portal in the First Story

n.C	Result	a	Cc (kg)	Cs (kg)	Ts (kg)	Pn (ton)	Mn (ton-m)
0.8	54.42	43.53	1321063.61	242982.54	262506.46	1301.54	775.01
0.7	47.62	38.09	1155930.66	242982.54	262506.46	1136.41	743.33
0.6	40.81	32.65	990797.71	242982.54	262506.46	971.27	702.66
0.5	34.01	27.21	825664.76	242982.54	262506.46	806.14	653.00
0.4	27.21	21.77	660531.80	242982.54	262506.46	641.01	594.36
0.3	20.41	16.33	495398.85	242982.54	262506.46	475.87	526.73

5. Pure bending

Using quadratic formula: $xa^2 + ya - z = 0$

$x = 0.85 \cdot f'c \cdot B = 30345$

$y = As' \cdot \varepsilon_{cu} \cdot Es - As \cdot fy = 131009.25$

$z = As' \cdot \varepsilon_{cu} \cdot Es \cdot \beta \cdot d' = -2077762.92$

$a = \frac{-y \pm \sqrt{y^2 - 4xz}}{2x} = 6.39 \text{ cm}$

$C = \frac{a}{\beta} = 7.99 \text{ cm}$

$\varepsilon_s = \frac{C-d}{C} \cdot \varepsilon_c = 0.00052$

$fs = \varepsilon_s \cdot E = 1064.82 \text{ kg/cm}^2$

$Cc = 0.85 \cdot f'c \cdot a \cdot B = 193996 \text{ kg}$

$Ts = As \cdot fs = 68510.46 \text{ kg}$

$Mn = Cc\left(h - \frac{a}{2}\right) + Ts(h - d') = 286.96 \text{ ton-m}$

$Pn = 0 \text{ ton}$

6. Pure tensile

$Pt = -(As + As')fy = -525.01 \text{ kg}$

$Mt = 0 \text{ ton-m}$

Hence, the final Pn and Mn values under each condition of the X direction in the first story are as follows.

Table 5.65 Final Pn and Mn Values of the X Portal in the First Story

Condition	Pn (ton)	Mn (ton-m)
Centric load	4127.37	0
Compression failure	2676.14	628.40
	2479.77	676.30
	2278.59	717.78
	2071.41	753.49
	1856.58	784.29
	1812.51	789.96
	1677.70	806.21
	1652.52	809.08
Balance	1631.81	811.41
Tension failure	1301.54	775.01
	1136.41	743.33
	971.27	702.66
	806.14	653.00
	641.01	594.36
	475.87	526.73
Pure bending	0	286.96
Pure tensile	-525.01	0

To analyze the Mn-Pn diagram, the ultimate moments (Mu) and ultimate axial loads (Pu) of every story must be derived from the ETABS model analysis. The values obtained can be seen as follows.

Table 5.66 Mu and Pu of Column of Both Portals in Every Story Group

Story	X Portal		Y Portal	
	Pu (ton)	Mu (ton-m)	Pu (ton)	Mu (ton-m)
15	71.09	44.17	70.95	31.97
12-14	338.30	49.94	337.55	27.11
9-11	614.59	57.88	614.59	30.31
6-8	901.25	79.20	901.25	34.44
2-5	1298.36	139.27	1298.36	57.50
1	1391.32	197.85	1391.32	65.05

With Mu of 197.85 ton-m and Pu of 1391.32 ton in the X portal in the first story, the Mn-Pn diagram analysis result is as follows.

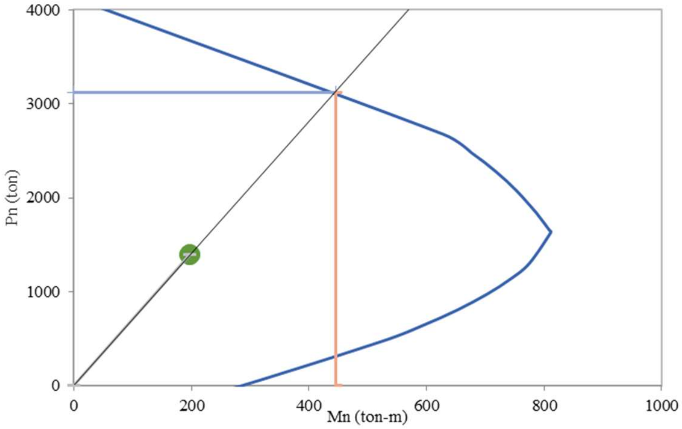


Figure 5.19 Mn-Pn Diagram Result of the X Portal in the First Story

From the graph, the Mn and Pn values obtained are as follows.

$\mathrm{Mn} = 445$ ton-m

$\mathrm{Pn} = 3118$ ton

The calculation process with Mn-Pn diagram continues for other stories, as well as for the Y portal. The conclusion of Mn and Pn values can be seen as follows.

Table 5.67 Mn and Pn of Column of Both Portals in Every Story Group

Story	X Portal		Y Portal	
	Pn (ton)	Mn (ton-m)	Pn (ton)	Mn (ton-m)
15	1210	760	1300	585
12-14	3080	455	3270	263
9-11	3388	320	3515	173
6-8	3440	303	3600	138
2-5	3305	355	3545	158
1	3118	445	3542	166

To confirm whether the nominal moment of the columns have completed the requirements of Strong Column Weak Beam (SCWB), the nominal moment of the columns must be larger than 1.2 times of the total nominal moment of both the left and right beams intersecting the column in each portal. For the X portal, the column intersects with beams BI1X on both sides, while for the Y portal the column intersects with beams BI1Y on both sides. Therefore, the nominal moment of beams previously obtained is incorporated into the SCWB analysis. The total nominal moment of the left and right beams is as follows.

Table 5.68 Total Mn of Beams BI1X in the X Portal

Story	Mn Left Beam	Mn Right Beam	ΣMn Beam	1.2 ΣMn Beam
15	364.70	364.70	729.41	875.29
14	709.13	537.27	1246.40	1495.67
13	709.13	537.27	1246.40	1495.67
12	709.13	537.27	1246.40	1495.67
11	880.29	708.96	1589.25	1907.10
10	880.29	708.96	1589.25	1907.10
9	880.29	708.96	1589.25	1907.10
8	1050.70	708.89	1759.60	2111.52
7	1050.70	708.89	1759.60	2111.52
6	1050.70	708.89	1759.60	2111.52
5	1050.70	708.89	1759.60	2111.52
4	1050.70	708.89	1759.60	2111.52
3	1050.70	708.89	1759.60	2111.52
2	1050.70	708.89	1759.60	2111.52
1	709.13	537.27	1246.40	1495.67

Table 5.69 Total Mn of Beams BI1Y in the Y Portal

Story	Mn Left Beam	Mn Right Beam	ΣMn Beam	1.2 ΣMn Beam
15	364.70	364.70	729.41	875.29
14	709.13	537.27	1246.40	1495.67
13	709.13	537.27	1246.40	1495.67
12	709.13	537.27	1246.40	1495.67
11	1050.70	708.89	1759.60	2111.52
10	1050.70	708.89	1759.60	2111.52
9	1050.70	708.89	1759.60	2111.52
8	1051.13	880.31	1931.44	2317.72
7	1051.13	880.31	1931.44	2317.72
6	1051.13	880.31	1931.44	2317.72
5	1051.13	880.31	1931.44	2317.72
4	1051.13	880.31	1931.44	2317.72
3	1051.13	880.31	1931.44	2317.72
2	1051.13	880.31	1931.44	2317.72
1	880.12	537.24	1417.36	1700.83

Furthermore, the total nominal moment of the upper story and lower story columns is as follows.

Table 5.70 Total Mn of Columns in the X Portal

Story	Mn Upper Column	Mn Lower Column	ΣMn Column
15	0	7455.60	7455.60
14	7455.60	4463.55	11919.15
13	4463.55	4463.55	8927.10
12	4463.55	4463.55	8927.10

11	4463.55	3139.20	7602.75
10	3139.20	3139.20	6278.40
9	3139.20	3139.20	6278.40
8	3139.20	2972.43	6111.63
7	2972.43	2972.43	5944.86
6	2972.43	2972.43	5944.86
5	2972.43	3482.55	6454.98
4	3482.55	3482.55	6965.10
3	3482.55	3482.55	6965.10
2	3482.55	3482.55	6965.10
1	3482.55	4365.45	7848.00

Table 5.71 Total Mn of Columns in the Y Portal

Story	Mn Upper Column	Mn Lower Column	ΣMn Column
15	0	5738.85	5738.85
14	5738.85	2580.03	8318.88
13	2580.03	2580.03	5160.06
12	2580.03	2580.03	5160.06
11	2580.03	1697.13	4277.16
10	1697.13	1697.13	3394.26
9	1697.13	1697.13	3394.26
8	1697.13	1353.78	3050.91
7	1353.78	1353.78	2707.56
6	1353.78	1353.78	2707.56
5	1353.78	1549.98	2903.76
4	1549.98	1549.98	3099.96
3	1549.98	1549.98	3099.96
2	1549.98	1549.98	3099.96
1	1549.98	1628.46	3178.44

Finally, the Mn beam and column comparison analysis for the SCWB requirement can be seen as follows.

Table 5.72 SCWB Analysis in the X Portal

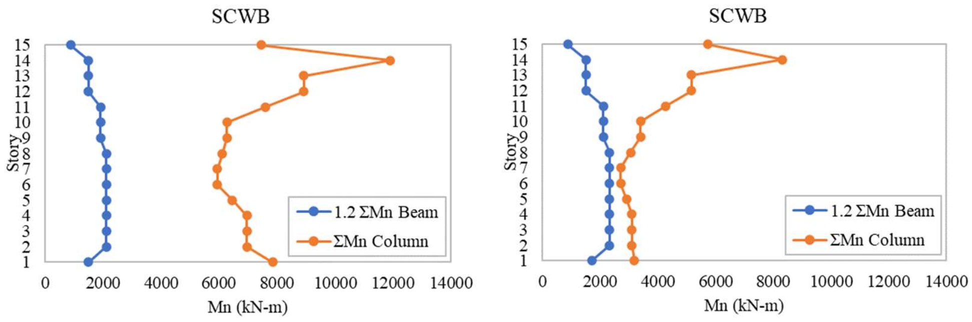
Story	1.2 ΣMn Beam	ΣMn Column	Ratio	Check
15	875.29	7455.60	8.52	OK
14	1495.67	11919.15	7.97	OK
13	1495.67	8927.10	5.97	OK
12	1495.67	8927.10	5.97	OK
11	1907.10	7602.75	3.99	OK
10	1907.10	6278.40	3.29	OK
9	1907.10	6278.40	3.29	OK
8	2111.52	6111.63	2.89	OK
7	2111.52	5944.86	2.82	OK
6	2111.52	5944.86	2.82	OK
5	2111.52	6454.98	3.06	OK

4	2111.52	6965.10	3.30	OK
3	2111.52	6965.10	3.30	OK
2	2111.52	6965.10	3.30	OK
1	1495.67	7848.00	5.25	OK

Table 5.73 SCWB Analysis in the Y Portal

Story	1.2 ΣMn Beam	ΣMn Column	Ratio	Check
15	875.29	5738.85	6.56	OK
14	1495.67	8318.88	5.56	OK
13	1495.67	5160.06	3.45	OK
12	1495.67	5160.06	3.45	OK
11	2111.52	4277.16	2.03	OK
10	2111.52	3394.26	1.61	OK
9	2111.52	3394.26	1.61	OK
8	2317.72	3050.91	1.32	OK
7	2317.72	2707.56	1.17	OK
6	2317.72	2707.56	1.17	OK
5	2317.72	2903.76	1.25	OK
4	2317.72	3099.96	1.34	OK
3	2317.72	3099.96	1.34	OK
2	2317.72	3099.96	1.34	OK
1	1700.83	3178.44	1.87	OK

The SCWB analysis is made into a graph/diagram as follows.



(a) (b)

Figure 5.20 SCWB Analysis of the (a) X Portal and (b) Y Portal

Conclusively, all the nominal moment of the columns has fulfilled the SCWB requirement which is to surpass 1.2 the value of the nominal moment of the beams. Therefore, the flexural reinforcement of 24D32 can be used in all stories.

Meanwhile, the shear reinforcement design of the columns differs in the first story from the other stories because plastic joint happens in the first story. Therefore, the shear reinforcement design of the first story also differs from the other stories. With the length of the column of 4 m, the calculation of the ultimate shear force (Vu) in the X direction of the 15th story as an example is as follows.

$$\mathrm{Vu} = \frac{\mathrm{Muc_{15} + Muc_{14}}}{\mathrm{L_{column}}} = 23.53 \text{ ton}$$

Meanwhile, the calculation of the earthquake shear force (Ve) in the X direction of the 15th story as an example is as follows.

$$\mathrm{Mprc_{15}} = 1.0(\mathrm{Mpr^{-}b_{15}} + \mathrm{Mpr^{+}b_{15}}) = 902.20 \text{ ton-m}$$

$$\mathrm{Mprc_{14}} = 0.5(\mathrm{Mpr^{-}b_{14}} + \mathrm{Mpr^{+}b_{14}}) = 773.13 \text{ ton-m}$$

$$\mathrm{Ve} = \frac{\mathrm{Mprc_{15} + Mprc_{14}}}{\mathrm{L_{column}}} = 418.83 \text{ ton}$$

The maximum shear force between Vu and Ve is the value of Ve, so the value of V used in the X direction of the 15th story is 418.83 ton.

The calculation results for both X and Y directions are compiled in the following tables.

Table 5.74 Column Shear Forces of Each Story in the X Direction

Story	Pu (ton)	E-X						
		Muc (ton-m)	Mpr-b (ton-m)	Mpr+b (ton-m)	Mprc (ton-m)	Vu (ton)	Ve (ton)	V used (ton)
15	71.09	44.17	451.10	451.10	902.20	23.53	418.83	418.83
14	338.30	49.94	880.12	666.14	773.13	24.97	386.56	386.56
13	338.30	49.94	880.12	666.14	773.13	24.97	386.56	386.56
12	338.30	49.94	880.12	666.14	773.13	26.95	439.97	439.97
11	614.59	57.88	1093.14	880.32	986.73	28.94	493.37	493.37
10	614.59	57.88	1093.14	880.32	986.73	28.94	493.37	493.37
9	614.59	57.88	1093.14	880.32	986.73	34.27	519.72	519.72
8	901.25	79.20	1304.00	880.31	1092.16	39.60	546.08	546.08
7	901.25	79.20	1304.00	880.31	1092.16	39.60	546.08	546.08
6	901.25	79.20	1304.00	880.31	1092.16	54.62	546.08	546.08
5	1298.36	139.27	1304.00	880.31	1092.16	69.63	546.08	546.08
4	1298.36	139.27	1304.00	880.31	1092.16	69.63	546.08	546.08
3	1298.36	139.27	1304.00	880.31	1092.16	69.63	546.08	546.08
2	1298.36	139.27	1304.00	880.31	1092.16	84.28	466.32	466.32
1	1391.32	197.85	880.12	666.14	773.13	Design with plastic joint		

Table 5.75 Column Shear Forces of Each Story in the Y Direction

Story	Pu (ton)	E-Y						
		Muc (ton-m)	Mpr-b (ton-m)	Mpr+b (ton-m)	Mprc (ton-m)	Vu (ton)	Ve (ton)	V used (ton)
15	70.95	31.97	451.10	451.10	902.20	14.77	418.83	418.83
14	337.55	27.11	880.12	666.14	773.13	13.56	386.56	386.56
13	337.55	27.11	880.12	666.14	773.13	13.56	386.56	386.56
12	337.55	27.11	880.12	666.14	773.13	14.36	466.32	466.32
11	614.59	30.31	1304.00	880.31	1092.16	15.16	546.08	546.08
10	614.59	30.31	1304.00	880.31	1092.16	15.16	546.08	546.08
9	614.59	30.31	1304.00	880.31	1092.16	16.19	573.01	573.01
8	901.25	34.44	1305.73	1094.01	1199.87	17.22	599.93	599.93
7	901.25	34.44	1305.73	1094.01	1199.87	17.22	599.93	599.93
6	901.25	34.44	1305.73	1094.01	1199.87	22.99	599.93	599.93
5	1298.36	57.50	1305.73	1094.01	1199.87	28.75	599.93	599.93
4	1298.36	57.50	1305.73	1094.01	1199.87	28.75	599.93	599.93
3	1298.36	57.50	1305.73	1094.01	1199.87	28.75	599.93	599.93
2	1298.36	57.50	1305.73	1094.01	1199.87	30.64	519.73	519.73
1	1391.32	65.05	1092.02	666.06	879.04	Design with plastic joint		

The following analysis is for the shear reinforcement design of the columns in the first story.

$f'c = 35\ \text{MPa} = 357\ \text{kg/cm}^2$

$fy = 400\ \text{MPa} = 4080\ \text{kg/cm}^2$

$Ht = 1200\ \text{mm} = 120\ \text{cm}$

$B = 1000\ \text{mm} = 100\ \text{cm}$

$\text{Concrete cover (Sb)} = 40\ \text{mm} = 4\ \text{cm}$

$D_{\text{flexural}} = 32\ \text{mm} = 3.2\ \text{cm}$

$D_{\text{shear}} = 10\ \text{mm} = 1\ \text{cm}$

$H_{\text{upper beam}} = 900\ \text{mm} = 90\ \text{cm}$

$H_{\text{lower beam}} = 0\ \text{mm} = 0\ \text{cm}$ (there is no beam underneath the first story)

$h_{\text{X direction}} = B - Sb - D_{\text{shear}} - \left(\frac{D_{\text{flexural}}}{2}\right) = 93.4\ \text{cm}$

$h_{\text{Y direction}} = Ht - Sb - D_{\text{shear}} - \left(\frac{D_{\text{flexural}}}{2}\right) = 113.4\ \text{cm}$

$L_{\text{column}} = 4\ \text{m} \rightarrow Ln_{\text{column}} = L_{\text{column}} - \frac{H_{\text{upper beam}}}{2} - \frac{H_{\text{lower beam}}}{2} = 3.55\ \text{m}$

$Hx = B - 2 \cdot Sb = 92\ \text{cm}$

$\mathrm{Hy} = \mathrm{Ht} - 2 \cdot \mathrm{Sb} = 112\ \mathrm{cm}$

$\mathrm{Bc} = \mathrm{B} - 2 \cdot \mathrm{Sb} - \mathrm{D_{shear}} = 91\ \mathrm{cm}$

$\mathrm{Hc} = \mathrm{Ht} - 2 \cdot \mathrm{Sb} - \mathrm{D_{shear}} = 111\ \mathrm{cm}$

$\frac{\mathrm{f'c}}{\mathrm{fy}} = 0.088$

$\mathrm{Ad} = \mathrm{As_{shear}} = \frac{1}{4} \cdot \pi \cdot \mathrm{D_{shear}}^2 = 0.785\ \mathrm{cm}^2$

$\mathrm{Ag} = \mathrm{Ht} \times \mathrm{B} = 12000\ \mathrm{cm}^2$

$\mathrm{Ach} = \mathrm{Hx} \times \mathrm{Hy} = 10304\ \mathrm{cm}^2$

$\frac{\mathrm{Ag}}{\mathrm{Ach}} = 1.165$

The plastic joint area is marked/named as Lo. The length of the plastic joint (Lo) must not surpass the following values as a requirement according to SNI 2847:2019.

1. $\frac{1}{6} \cdot \mathrm{Ln} = 59.17\ \mathrm{cm}$
2. $\text{Maximum column dimension} = \mathrm{Ht} = 120\ \mathrm{cm}$
3. $450\ \mathrm{mm} = 45\ \mathrm{cm}$

Therefore, the length of the plastic joint area (Lo) is used 120 cm.

The shear reinforcement is then designed in the plastic joint area (Lo). The spacing checking according to SNI 2847:2019 is as follows.

1. $\frac{1}{4} \cdot \text{Minimum column dimension} = \frac{1}{4} \cdot \mathrm{B} = 25\ \mathrm{cm}$
2. $\frac{1}{6} \cdot \text{Minimum } \mathrm{D_{flexural}} = 19.2\ \mathrm{cm}$
3. $100\ \mathrm{mm} = 10\ \mathrm{cm}$

Therefore, the spacing (s) used is 10 cm.

The shear reinforcement design in the X direction of the plastic joint area (Lo) according to SNI 2847:2019 is as follows. In the X direction, the Bc value used is the value of Hx of 92 cm.

$\mathrm{Ash_1} = 0.3 \left(\mathrm{s} \cdot \mathrm{Bc} \cdot \frac{\mathrm{f'c}}{\mathrm{fyt}} \right) \left(\frac{\mathrm{Ag}}{\mathrm{Ach}} - 1 \right) = 3.98\ \mathrm{cm}^2 \ \rightarrow\ \mathrm{n_1} = \frac{\mathrm{Ash_1}}{\mathrm{Ad}} = 5.06 \approx 6$

$\mathrm{Ash_2} = 0.9 \left(\mathrm{s} \cdot \mathrm{Bc} \cdot \frac{\mathrm{f'c}}{\mathrm{fyt}} \right) = 7.25\ \mathrm{cm}^2 \ \rightarrow\ \mathrm{n_2} = \frac{\mathrm{Ash_2}}{\mathrm{Ad}} = 9.22 \approx 10$

Because the larger n value is 10, the number of reinforcements (n) used in the X direction is 10. The spacing between the legs of the stirrups or shear reinforcements in the X direction is as follows.

$$x_1 = \frac{\left(B-2\cdot Sb-2\cdot\frac{D_{shear}}{2}\right)}{n-1} = 10.11 \text{ cm} < 35 \text{ cm (OK)}$$

Meanwhile, the shear reinforcement design in the Y direction of the plastic joint area (Lo) according to SNI 2847:2019 is as follows. In the Y direction, the Bc value used is the value of Hy of 112 cm.

$$Ash_1 = 0.3\left(s \cdot Bc \cdot \frac{f'c}{fyt}\right)\left(\frac{Ag}{Ach}-1\right) = 4.84 \text{ cm}^2 \rightarrow n_1 = \frac{Ash_1}{Ad} = 6.16 \approx 7$$

$$Ash_2 = 0.9\left(s \cdot Bc \cdot \frac{f'c}{fyt}\right) = 8.82 \text{ cm}^2 \rightarrow n_2 = \frac{Ash_2}{Ad} = 11.23 \approx 12$$

Because the larger n value is 12, the number of reinforcements (n) used in the Y direction is 12. The spacing between the legs of the stirrups or shear reinforcements in the Y direction is as follows.

$$x_1 = \frac{\left(B-2\cdot Sb-2\cdot\frac{D_{shear}}{2}\right)}{n-1} = 8.27 \text{ cm} < 35 \text{ cm (OK)}$$

The column shear reinforcement used in the plastic joint area (Lo) of the first story is taken the maximum n value with the minimum s value, which is 10D10-100 mm for the X direction and 12D10-100 mm for the Y direction. However, as the number of reinforcements used in the X and Y portals do not reach the values of 10 or 12, the shear reinforcements are designed as closed stirrup overlap. The illustration is attached in the appendix.

Furthermore, the shear reinforcement design in the X direction outside of the plastic joint area (outside of Lo) according to SNI 2847:2019 is as follows. The spacing is first checked so that it does not surpass h/2.

$$\frac{h_{X\ direction}}{2} = 46.70 \text{ cm}$$

As the columns of the first story are considered a plasitic joint area along the span length, the spacing (s) must not exceed 15 cm. Therefore, the spacing (s) used is 15 cm. The number of reinforcements in the X direction is estimated to be 4. The spacing between the legs of the stirrups or shear reinforcements is as follows.

$$x_1 = \frac{\left(B-2\cdot Sb-2\cdot\frac{D_{shear}}{2}\right)}{n-1} = 30.33 \text{ cm} < 35 \text{ cm (OK)}$$

Meanwhile, the shear reinforcement design in the Y direction outside of the plastic joint area (outside of Lo) according to SNI 2847:2019 is as follows. The spacing is first checked so that it does not surpass h/2.

$\frac{h_{\text{Y direction}}}{2} = 56.70 \text{ cm}$

As the columns of the first story are considered a plasitic joint area along the span length, the spacing (s) must not exceed 15 cm. Therefore, the spacing (s) used is 15 cm. The number of reinforcements in the Y direction is estimated to be 4. The spacing between the legs of the stirrups or shear reinforcements is as follows.

$x_1 = \frac{\left(B - 2 \cdot Sb - 2 \cdot \frac{D_{shear}}{2}\right)}{n-1} = 30.33 \text{ cm} < 35 \text{ cm (OK)}$

The column shear reinforcement used outside of the plastic joint area (outside of Lo) of the first story in the X and Y directions is taken the maximum n value with the minimum s value, which is 4D10-150 mm.

Another shear reinforcement analysis must be carried out for the other stories. The following analysis is for the shear reinforcement design of the columns in stories 2-5 as an example.

$f'c = 35 \text{ MPa} = 357 \text{ kg/cm}^2$

$fy = 400 \text{ MPa} = 4080 \text{ kg/cm}^2$

$Ht = 1200 \text{ mm} = 120 \text{ cm}$

$B = 1000 \text{ mm} = 100 \text{ cm}$

$\text{Concrete cover (Sb)} = 40 \text{ mm} = 4 \text{ cm}$

$D_{flexural} = 32 \text{ mm} = 3.2 \text{ cm}$

$D_{shear} = 10 \text{ mm} = 1 \text{ cm}$

$H_{\text{upper beam}} = 900 \text{ mm} = 90 \text{ cm}$

$H_{\text{lower beam}} = 900 \text{ mm} = 90 \text{ cm}$

$h_{\text{X direction}} = B - Sb - D_{shear} - \left(\frac{D_{flexural}}{2}\right) = 93.4 \text{ cm}$

$h_{\text{Y direction}} = Ht - Sb - D_{shear} - \left(\frac{D_{flexural}}{2}\right) = 113.4 \text{ cm}$

$L_{column} = 4 \text{ m} \rightarrow Ln_{column} = L_{column} - \frac{H_{\text{upper beam}}}{2} - \frac{H_{\text{lower beam}}}{2} = 3.55 \text{ m}$

$Ad = As_{shear} = \frac{1}{4} \cdot \pi \cdot {D_{shear}}^2 = 0.785 \text{ cm}^2$

$\lambda = 1$ (Regular concrete)

$\mathrm{Nu} = \mathrm{Pu} = 1298.36\ \mathrm{ton}$ (from ETABS)

$\mathrm{Ag} = \mathrm{Ht} \times \mathrm{B} = 12000\ \mathrm{cm}^2$

$\frac{\mathrm{Nu}}{\mathrm{Ag}} = 10.608$ MPa

The plastic joint area is marked/named as Lo. The length of the plastic joint (Lo) must not surpass the following values as a requirement according to SNI 2847:2019.

1. $\frac{1}{6} \cdot \mathrm{Ln} = 59.17\ \mathrm{cm}$
2. $\mathrm{Maximum\ column\ dimension} = \mathrm{Ht} = 120\ \mathrm{cm}$
3. $450\ \mathrm{mm} = 45\ \mathrm{cm}$

Therefore, the length of the plastic joint area (Lo) is used 120 cm.

The shear reinforcement design in the plastic joint area (Lo) in the X direction for stories 2-5 is as follows.

$\mathrm{V_{used}} = 546.08\ \mathrm{ton}$

$\varphi = 0.75$

$\mathrm{Vn} = \frac{\mathrm{V_{used}}}{\varphi} = 728.10\ \mathrm{ton}$

$\mathrm{Vc} = 0.17\left(1 + \frac{\mathrm{Nu}}{14 \cdot \mathrm{Ag}}\right)\lambda \cdot \sqrt{\mathrm{f'c}} \cdot \mathrm{B} \cdot \mathrm{h_{X\ direction}} = 1651087.13\ \mathrm{N} = 168.31\ \mathrm{ton}$

$\mathrm{Vs} = \mathrm{Vn} - \mathrm{Vc} = 559.80\ \mathrm{ton}$

The number of reinforcements (n) used is estimated to be 10 legs. The spacing (s) must be checked to be used. The spacing (s) checking is as follows.

$\mathrm{Av} = \mathrm{n} \cdot \mathrm{Ad} = 7.85\ \mathrm{cm}^2$

$\mathrm{s_{max}} = \frac{\mathrm{Av \cdot fy \cdot h_{X\ direction}}}{\mathrm{Vs}} = 5.35\ \mathrm{cm}$

The spacing checking in the X direction according to SNI 2847:2019 is as follows.

1. $8 \cdot \mathrm{Minimum\ D_{flexural}} = 25.6\ \mathrm{cm}$
2. $24 \cdot \mathrm{D_{shear}} = 24\ \mathrm{cm}$
3. $\frac{1}{2} \cdot \mathrm{Minimum\ column\ dimension} = \frac{1}{2} \cdot \mathrm{B} = 50\ \mathrm{cm}$
4. $300\ \mathrm{mm} = 30\ \mathrm{cm}$

As the spacing must be less than 24 cm and the maximum spacing is 5.35 cm, the spacing (s) used is 5 cm. The spacing between the legs of the stirrups or shear reinforcements in the X direction is as follows.

$$\mathrm{x_1} = \frac{\left(\mathrm{B - 2 \cdot Sb - 2 \cdot \frac{D_{shear}}{2}}\right)}{\mathrm{n-1}} = 10.11\ \mathrm{cm} < 35\ \mathrm{cm\ (OK)}$$

Meanwhile, the shear reinforcement design in the plastic joint area (Lo) in the Y direction for stories 2-5 is as follows.

$$\mathrm{V_{used}} = 599.93\ \mathrm{ton}$$

$$\varphi = 0.75$$

$$\mathrm{Vn} = \frac{\mathrm{V_{used}}}{\varphi} = 799.91\ \mathrm{ton}$$

$$\mathrm{Vc} = 0.17\left(1 + \frac{\mathrm{Nu}}{14 \cdot \mathrm{Ag}}\right)\lambda \cdot \sqrt{\mathrm{f'c}} \cdot \mathrm{B} \cdot \mathrm{h_{Y\ direction}} = 2405566.78\ \mathrm{N} = 245.22\ \mathrm{ton}$$

$$\mathrm{Vs} = \mathrm{Vn} - \mathrm{Vc} = 554.70\ \mathrm{ton}$$

The number of reinforcements (n) used is estimated to be 10 legs. The spacing (s) must be checked to be used. The spacing (s) checking is as follows.

$$\mathrm{Av} = \mathrm{n} \cdot \mathrm{Ad} = 7.85\ \mathrm{cm}^2$$

$$\mathrm{s_{max}} = \frac{\mathrm{Av \cdot fy \cdot h_{Y\ direction}}}{\mathrm{Vs}} = 6.55\ \mathrm{cm}$$

The spacing checking in the Y direction according to SNI 2847:2019 is as follows.

1. $8 \cdot \text{Minimum } \mathrm{D_{flexural}} = 25.6\ \mathrm{cm}$
2. $24 \cdot \mathrm{D_{shear}} = 24\ \mathrm{cm}$
3. $\frac{1}{2} \cdot \text{Minimum column dimension} = \frac{1}{2} \cdot \mathrm{B} = 50\ \mathrm{cm}$
4. $300\ \mathrm{mm} = 30\ \mathrm{cm}$

As the spacing must be less than 24 cm and the maximum spacing is 6.55 cm, the spacing (s) used is 6 cm. The spacing between the legs of the stirrups or shear reinforcements in the Y direction is as follows.

$$\mathrm{x_1} = \frac{\left(\mathrm{B - 2 \cdot Sb - 2 \cdot \frac{D_{shear}}{2}}\right)}{\mathrm{n-1}} = 10.11\ \mathrm{cm} < 35\ \mathrm{cm\ (OK)}$$

The column shear reinforcement used in the plastic joint area (Lo) of stories 2-5 in the X and Y directions is taken the maximum n value with the minimum s value, which is 10D10-50 mm. However, as the number of reinforcements used in the X and Y portals do not reach the values of 10, the shear reinforcements are designed as closed stirrup overlap. The illustration is attached in the appendix.

Furthermore, the shear reinforcement design in the X direction outside of the plastic joint area (outside of Lo) according to SNI 2847:2019 is as follows. The spacing is first checked so that it does not surpass h/2.

$$\frac{\mathrm{h_{X\ direction}}}{2} = 46.70\ \mathrm{cm}$$

As the spacing (s) does not need to be less than 35 cm, the spacing (s) used is 35 cm. The number of reinforcements in the X direction is estimated to be 4. The spacing between the legs of the stirrups or shear reinforcements is as follows.

$$\mathrm{x_1} = \frac{\left(\mathrm{B-2\cdot Sb-2\cdot\frac{D_{shear}}{2}}\right)}{\mathrm{n-1}} = 30.33\ \mathrm{cm} < 35\ \mathrm{cm\ (OK)}$$

Meanwhile, the shear reinforcement design in the Y direction outside of the plastic joint area (outside of Lo) according to SNI 2847:2019 is as follows. The spacing is first checked so that it does not surpass h/2.

$$\frac{\mathrm{h_{Y\ direction}}}{2} = 56.70\ \mathrm{cm}$$

As the spacing (s) does not need to be less than 35 cm, the spacing (s) used is 35 cm. The number of reinforcements in the Y direction is estimated to be 4. The spacing between the legs of the stirrups or shear reinforcements is as follows.

$$\mathrm{x_1} = \frac{\left(\mathrm{B-2\cdot Sb-2\cdot\frac{D_{shear}}{2}}\right)}{\mathrm{n-1}} = 30.33\ \mathrm{cm} < 35\ \mathrm{cm\ (OK)}$$

The column shear reinforcement used outside of the plastic joint area (outside of Lo) of stories 2-5 in the X and Y directions is taken the maximum n value with the minimum s value, which is 4D10-350 mm.

Conclusively, the shear reinforcements calculation results within and outside of the plastic joint area (Lo) for both X and Y directions are compiled in the following table.

Table 5.76 Column Shear Reinforcement of Each Story in Both Directions

Story	Lo			Outside of Lo	
	Length (mm)	X Direction (mm)	Y Direction (mm)	X Direction (mm)	Y Direction (mm)
15	1200	10D10-50	10D10-50	4D10-350	4D10-350
12-14	1200	10D10-50	10D10-50	4D10-350	4D10-350

9-11	1200	10D10-50	10D10-50	4D10-350	4D10-350
6-8	1200	10D10-50	10D10-50	4D10-350	4D10-350
2-5	1200	10D10-50	10D10-50	4D10-350	4D10-350
1	1200	10D10-100	12D10-100	4D10-150	4D10-150

The next analysis is carried out for the beam-column joint (BCJ) of each story. The beam-column joint plays an important role in the stability of the structure. Priestley & Paulay (1992) stated that the main problems in the beam-column joints are as follows.

1. Horizontal and vertical shear forces may be several times greater than the shear force at adjacent beams and columns.
2. Joint stress problems arise due to the combination of compression and tension within the reinforcement line.

As beam-column joint is considered a plastic joint, the design of BCJ is similar to the shear reinforcement design in the first story. The BCJ design of the first story as an example is as follows.

$f'c = 35\ \text{MPa} = 357\ \text{kg/cm}^2$

$fy = 400\ \text{MPa} = 4080\ \text{kg/cm}^2$

$\text{Ht} = 1200\ \text{mm} = 120\ \text{cm}$

$\text{B} = 1000\ \text{mm} = 100\ \text{cm}$

$\text{Concrete cover (Sb)} = 40\ \text{mm} = 4\ \text{cm}$

$\text{D}_{\text{flexural}} = 32\ \text{mm} = 3.2\ \text{cm}$

$\text{D}_{\text{shear}} = 10\ \text{mm} = 1\ \text{cm}$

$\text{H}_{\text{upper beam}} = 900\ \text{mm} = 90\ \text{cm}$

$\text{H}_{\text{lower beam}} = 0\ \text{mm} = 0\ \text{cm}$ (there is no beam underneath the first story)

$\text{h}_{\text{X direction}} = \text{B} - \text{Sb} - \text{D}_{\text{shear}} - \left(\frac{\text{D}_{\text{flexural}}}{2}\right) = 93.4\ \text{cm}$

$\text{h}_{\text{Y direction}} = \text{Ht} - \text{Sb} - \text{D}_{\text{shear}} - \left(\frac{\text{D}_{\text{flexural}}}{2}\right) = 113.4\ \text{cm}$

$\text{L}_{\text{column}} = 4\ \text{m} \rightarrow \text{Ln}_{\text{column}} = \text{L}_{\text{column}} - \frac{\text{H}_{\text{upper beam}}}{2} - \frac{\text{H}_{\text{lower beam}}}{2} = 3.55\ \text{m}$

$\text{Hx} = \text{B} - 2 \cdot \text{Sb} = 92\ \text{cm}$

$\text{Hy} = \text{Ht} - 2 \cdot \text{Sb} = 112\ \text{cm}$

$\text{Bc} = \text{B} - 2 \cdot \text{Sb} - \text{D}_{\text{shear}} = 91\ \text{cm}$

$\mathrm{Hc} = \mathrm{Ht} - 2 \cdot \mathrm{Sb} - \mathrm{D_{shear}} = 111\ \mathrm{cm}$

$\frac{\mathrm{f'c}}{\mathrm{fy}} = 0.088$

$\mathrm{Ad} = \mathrm{As_{shear}} = \frac{1}{4} \cdot \pi \cdot \mathrm{D_{shear}}^2 = 0.785\ \mathrm{cm}^2$

$\mathrm{Ag} = \mathrm{Ht} \times \mathrm{B} = 12000\ \mathrm{cm}^2$

$\mathrm{Ach} = \mathrm{Hx} \times \mathrm{Hy} = 10304\ \mathrm{cm}^2$

$\frac{\mathrm{Ag}}{\mathrm{Ach}} = 1.165$

The plastic joint area is marked/named as Lo. The length of the plastic joint (Lo) must not surpass 150 mm as a requirement according to SNI 2847:2019. After further consideration, the spacing (s) used is 10 cm.

The BCJ design in the X direction according to SNI 2847:2019 is as follows. In the X direction, the Bc value used is the value of Hx of 92 cm.

$\mathrm{Ash_1} = 0.3 \left(\mathrm{s} \cdot \mathrm{Bc} \cdot \frac{\mathrm{f'c}}{\mathrm{fyt}}\right) \left(\frac{\mathrm{Ag}}{\mathrm{Ach}} - 1\right) = 3.98\ \mathrm{cm}^2 \rightarrow \mathrm{n_1} = \frac{\mathrm{Ash_1}}{\mathrm{Ad}} = 5.06 \approx 6$

$\mathrm{Ash_2} = 0.9 \left(\mathrm{s} \cdot \mathrm{Bc} \cdot \frac{\mathrm{f'c}}{\mathrm{fyt}}\right) = 7.25\ \mathrm{cm}^2 \rightarrow \mathrm{n_1} = \frac{\mathrm{Ash_2}}{\mathrm{Ad}} = 9.22 \approx 10$

Because the larger n value is 10, the number of reinforcements (n) used in the X direction is 10. The spacing between the legs in the X direction is as follows.

$\mathrm{x_1} = \frac{\left(\mathrm{B} - 2 \cdot \mathrm{Sb} - 2 \cdot \frac{\mathrm{D_{shear}}}{2}\right)}{\mathrm{n} - 1} = 10.11\ \mathrm{cm} < 35\ \mathrm{cm}\ \mathrm{(OK)}$

Meanwhile, the BCJ design in the Y direction according to SNI 2847:2019 is as follows. In the Y direction, the Bc value used is the value of Hy of 112 cm.

$\mathrm{Ash_1} = 0.3 \left(\mathrm{s} \cdot \mathrm{Bc} \cdot \frac{\mathrm{f'c}}{\mathrm{fyt}}\right) \left(\frac{\mathrm{Ag}}{\mathrm{Ach}} - 1\right) = 4.84\ \mathrm{cm}^2 \rightarrow \mathrm{n_1} = \frac{\mathrm{Ash_1}}{\mathrm{Ad}} = 6.16 \approx 7$

$\mathrm{Ash_2} = 0.9 \left(\mathrm{s} \cdot \mathrm{Bc} \cdot \frac{\mathrm{f'c}}{\mathrm{fyt}}\right) = 8.82\ \mathrm{cm}^2 \rightarrow \mathrm{n_2} = \frac{\mathrm{Ash_2}}{\mathrm{Ad}} = 11.23 \approx 12$

Because the larger n value is 12, the number of reinforcements (n) used in the Y direction is 12. The spacing between the legs in the Y direction is as follows.

$\mathrm{x_1} = \frac{\left(\mathrm{B} - 2 \cdot \mathrm{Sb} - 2 \cdot \frac{\mathrm{D_{shear}}}{2}\right)}{\mathrm{n} - 1} = 8.27\ \mathrm{cm} < 35\ \mathrm{cm}\ \mathrm{(OK)}$

The beam-column joint reinforcement of the first story used in the X direction is 10D10-100 mm, whilw in the Y direction is 12D10-100 mm. The analysis continues for each story. The calculation results are as follows.

Table 5.77 Beam-Column Joint Reinforcement of Each Story in Both Directions

Story	BCJ in X Direction (mm)	BCJ in Y Direction (mm)
15	10D10-50	10D10-50
12-14	10D10-50	10D10-50
9-11	10D10-50	10D10-50
6-8	10D10-50	10D10-50
2-5	10D10-50	10D10-50
1	10D10-100	12D10-100

The beam-column joint shear stress in all BCJ of each story in both X and Y directions must then be checked if the reinforcements have fulfilled the requirements of SNI 2847:2019. The example of the shear stress analysis for the BCJ on the first story in the X direction is as follows.

$\mathrm{f'c} = 35\ \mathrm{MPa} = 357\ \mathrm{kg/cm^2}$

$\varphi = 0.7$

As both the left and right beams in the X portal are BI1X, the data for beams is as follows.

$\mathrm{d_{left\ beam}} = \mathrm{d_{right\ beam}} = \mathrm{Sb} + \mathrm{D_{shear}} + \frac{\mathrm{D_{flexural}}}{2} = 6.4\ \mathrm{cm}$

$\mathrm{B_{left\ beam}} = \mathrm{B_{right\ beam}} = 45\ \mathrm{cm}$

$\mathrm{Ht_{left\ beam}} = \mathrm{Ht_{right\ beam}} = 90\ \mathrm{cm}$

$\mathrm{h_{left\ beam}} = \mathrm{h_{right\ beam}} = \mathrm{Ht} - \mathrm{d} = 83.6\ \mathrm{cm}$

$\mathrm{H_{column}} = 120\ \mathrm{cm}$

$\mathrm{B_{column}} = 100\ \mathrm{cm}$

$\mathrm{h_{upper\ column}} = \mathrm{h_{lower\ column}} = 4\ \mathrm{m} = 400\ \mathrm{cm}$

$\mathrm{Lb_{left\ beam}} = \mathrm{Lb_{right\ beam}} = 800\ \mathrm{cm}$

$\mathrm{Lbn_{left\ beam}} = \mathrm{Lbn_{right\ beam}} = \mathrm{Lb} - 2\left(\frac{1}{2} \cdot \mathrm{B_{column}}\right) = 700\ \mathrm{cm}$

$\mathrm{a_{left\ beam}} = 6.99\ \mathrm{cm}$ (from $\mathrm{Mpr^{-}}$ analysis of BI1X)

$\mathrm{a_{left\ beam}} = 5.73\ \mathrm{cm}$ (from $\mathrm{Mpr^{+}}$ analysis of BI1X)

$\mathrm{Mpr^{-}}_{\mathrm{left\ beam}} = 89.72$ ton-m

$\mathrm{Mpr^{+}}_{\mathrm{right\ beam}} = 67.90$ ton-m

The shear cross-section area (Aj) is as follows.

$$\mathrm{Aj} = \left(\frac{\mathrm{B_{column}} - \mathrm{B_{beam}}}{2} + \mathrm{B_{beam}}\right)\mathrm{H_{column}} = 8250\ \mathrm{cm}^2$$

The maximum nominal shear (Vn) is as follows.

$$\mathrm{Vn} = 1.7 \cdot \sqrt{\mathrm{f'c}} \cdot \mathrm{Aj} = 845.80\ \mathrm{ton}$$

The column shear strength ($\mathrm{V_{column}}$) is as follows.

$$\mathrm{V_{column}} = \frac{\varphi\left(\frac{\mathrm{Lb_{left}}}{\mathrm{Lbn_{left}}}\right)\mathrm{Mpr^{-}}_{\mathrm{left}} + \left(\frac{\mathrm{Lb_{right}}}{\mathrm{Lbn_{right}}}\right)\mathrm{Mpr^{+}}_{\mathrm{right}}}{\frac{1}{2}\left(\mathrm{h_{upper\ column}} + \mathrm{h_{lower\ column}}\right)} = 31.52\ \mathrm{ton}$$

Tension (Ts) from the left beam is as follows.

$$\mathrm{Ts} = \frac{\varphi \cdot \mathrm{Mpr^{-}}_{\mathrm{left}}}{\mathrm{h_{left}} - \frac{\mathrm{a_{left}}}{2}} = 78.40\ \mathrm{ton}$$

Compression (Cc) from the right beam is as follows.

$$\mathrm{Cc} = \frac{\varphi \cdot \mathrm{Mpr^{-}}_{\mathrm{right}}}{\mathrm{h_{right}} - \frac{\mathrm{a_{right}}}{2}} = 58.87\ \mathrm{ton}$$

The horizontal joint shear force (Vjh) is as follows.

$$\mathrm{Vjh} = \mathrm{Ts} + \mathrm{Cc} - \mathrm{V_{column}} = 105.75\ \mathrm{ton}$$

Because Vjh is less than Vn (Vjh < Vn), the column dimension is considered safe and can be used. Furthermore, the joint shear stress (τjh) and the maximum value are as follows.

$$\tau\mathrm{jh} = \frac{\mathrm{Vjh}}{\mathrm{Aj}} = 128.18\ \mathrm{ton/m^2}$$

$$\tau\mathrm{jh_{max}} = 1.7 \cdot \sqrt{\mathrm{f'c}} = 1025.85\ \mathrm{ton/m^2}$$

Because the value of τjh is less than the maximum value of τjh (τjh < τjh max), the column dimension is considered safe and can be used. This analysis is further carried out for all the joints in every story and in both X and Y directions. The analysis shows that all of the joints in all stories and in both X and Y directions are considered safe and that the column dimensions can be used.

5.6.4 Floor Plate Reinforcement Design

The initial data of the floor plate material properties is as follows.

f’c = 35 MPa

fy2 = 360 MPa

fy1 = 400 MPa

β = 0.80

$D_{flexural}$ = 10 mm

D_{shear} = 6 mm

h_{plate} = 0.14 m = 140 mm

Sb = 20 mm

ds = $Sb + 0.5(D_{flexural}) = 20 + 0.5(10) = 25$ mm

Ly = 3 m = 3000 mm

Lx = 4 m = 4000 mm

Lny = 2650 mm

Lnx = 3600 mm

From the data above, the shortest side net length Ln is obtained at Lnx, which is 2650 mm or 2.65 m.

The dead load excluding the additional load is calculated as follows.

$$\text{Dead load} = h_{plate} \times \gamma_{concrete} = 0.14 \times 2400 = 336\ \text{kg/m}^2$$

Meanwhile, the additional dead load of the floor plate can be seen in the following table.

Table 5.78 Additional Dead Load of Floor Plate

No	Component	Volume Weight		Thickness		Q	
		Value	Unit	Value	Unit	kg/m²	kN/m²
1	Partition					48.93	0.480
2	Sand	1600	kg/m³	0.05	m	80	0.785
3	Spec	21	kg/m²/cm thickness	3	cm	63	0.618
4	Ceramic					17	0.167
5	Mechanical & Electrical					30	0.294
6	Ceiling					9	0.088
7	Ceiling Hanger					5	0.049
Total Additional Dead Load						252.93	2.481

Hence, the ultimate load calculation of the floor plate is as follows.

Qd $= \text{Dead load} + \text{Additional dead load} = 336 + 252.93 = 588.93\ \text{kg/m}^2$

Ql $= 2.40\ \text{kN/m}^2 = 244.65\ \text{kg/m}^2$

Qu $= 1.2 \cdot \text{Qd} + 1.6 \cdot \text{Ql} = 1.2 \cdot 588.93 + 1.6 \cdot 244.65$

$= 1098.15\ \text{kg/m}^2 = 10.77\ \text{kN/m}^2$

The coefficient parameters used to calculate the moment values according to the table from PBI 1971 are as follows.

Lx/Ly = 4000/3000 = 1.333 ≈ 1.3

x_{Mtx-} = 69

x_{Mlx+} = 31

x_{Mty-} = 57

x_{Mly+} = 19

Lx in this case is the shortest side, while Ly is the longest. So, for the calculation of the moments, Lx is 3 m while Ly is 4 m. Hence, the calculation of the moments is as follows.

$Mtx^{-} = -0.001 \cdot Qu \cdot Lx^2 \cdot x_{Mtx-} = -0.001 \cdot 10.77 \cdot 3^2 \cdot 69 = -6.69$ kNm

$Mlx^{+} = 0.001 \cdot Qu \cdot Lx^2 \cdot x_{Mlx+} = 0.001 \cdot 10.77 \cdot 3^2 \cdot 31 = 3.01$ kNm

$Mty^{-} = -0.001 \cdot Qu \cdot Lx^2 \cdot x_{Mty-} = -0.001 \cdot 10.77 \cdot 3^2 \cdot 57 = -5.53$ kNm

$Mly^{+} = 0.001 \cdot Qu \cdot Lx^2 \cdot x_{Mly+} = 0.001 \cdot 10.77 \cdot 3^2 \cdot 19 = 1.84$ kNm

Furthermore, the reinforcement design is divided into four types for one floor plate, which are the support and middle span areas in the X direction, as well as the support and middle span areas in the Y direction. The following calculation shows the reinforcement design of the support area in the X direction.

1. Shear force calculation

   $Vu = 0.5 \cdot 1.15 \cdot Qu \cdot Ln = 0.5 \cdot 1.15 \cdot 10.77 \cdot 2.65 = 16.42$ kN

   $d = h - ds = 140 - 25 = 115$ mm

   $b_w = 1000$ mm

   $Vn = 0.17 \cdot \sqrt{f'c} \cdot b_w \cdot d = 0.17 \cdot \sqrt{35} \cdot 1000 \cdot 115 = 115659.36$ N

   $\varphi = 0.75$

   $\varphi Vn = 0.75 \cdot 115659.36 = 86744.52$ N $= 86.74$ kN

   Check:

   $\varphi Vn > Vu$

   86.74 kN $>$ 16.42 kN (SAFE, plate dimension can withstand shear force)

2. Moment calculation

   $Mu = 6.69$ kNm

   $\varphi = 0.90$

   $Mn = \frac{Mu}{\varphi} = \frac{6.69}{0.90} = 7.43$ kNm

$b = 1000\ \text{mm}$

$d = 115\ \text{mm}$

Using quadratic formula to determine the value of a

$Mn = 0.85 \cdot f'c \cdot a \cdot b \cdot \left(d - \frac{a}{2}\right)$

$7.43 = 0.85 \cdot 35 \cdot a \cdot 1000 \cdot \left(115 - \frac{a}{2}\right)$

From the quadratic formula, the value of a is obtained 2.19 mm.

3. Flexural reinforcement design

The As area is determined with the Ts = Cc formula.

$As_{\text{min}} = 0.002 \cdot Ag = 0.002 \cdot (1000 \cdot 115) = 230\ \text{mm}^2$

$As_{\text{needed}} = 0.85 \cdot f'c \cdot a \cdot \frac{b}{fy} = 0.85 \cdot 35 \cdot 2.19 \cdot \frac{1000}{360} = 181.28\ \text{mm}^2$

$As_{\text{balance}} = 0.85 \cdot \beta_1 \cdot \frac{f'c}{fy} \cdot \frac{600}{600+fy} \cdot b \cdot d$

$As_{\text{balance}} = 0.85 \cdot 0.80 \cdot \frac{35}{360} \cdot \frac{600}{600+360} \cdot 1000 \cdot 115 = 4751.74\ \text{mm}^2$

$As_{\text{max}} = 0.75 \cdot As_{\text{balance}} = 0.75 \cdot 4751.74 = 3563.80\ \text{mm}^2$

$As_{\text{used}} = 230\ \text{mm}^2$

$As_{1D} = \frac{1}{4} \cdot \pi \cdot D_{\text{flexural}}{}^2 = \frac{1}{4} \cdot \pi \cdot 10^2 = 78.54\ \text{mm}^2$

$s = As_{1D} \cdot \frac{b}{As_{\text{used}}} = 78.54 \cdot \frac{1000}{230} = 341.48\ \text{mm}$

$s_{\text{used}} = 200\ \text{mm}$

$As_{\text{reinforcement used}} = \frac{b}{s_{\text{used}}} \cdot As_{1D} = \frac{1000}{200} \cdot 78.54 = 392.70\ \text{mm}^2$

Check:

a. $s_{\text{used}} \leq 2d$

$200\ \text{mm} \leq 2 \cdot 115\ \text{mm}$

$200\ \text{mm} \leq 230\ \text{mm}$ (OK)

b. $s_{\text{used}} \leq 450\ \text{mm}$

$200\ \text{mm} \leq 450\ \text{mm}$ (OK)

Hence, it can be concluded that based on the calculations, the flexural reinforcement used for the support area of floor plate in the X direction is D10-200 mm.

4. Shear reinforcement design

$$As_{shear} = 0.002 \cdot b \cdot h = 0.002 \cdot 1000 \cdot 140 = 280\ mm^2$$

$$As_{1D} = \frac{1}{4} \cdot \pi \cdot D_{shear}{}^2 = \frac{1}{4} \cdot \pi \cdot 6^2 = 28.27\ mm^2$$

$$s = As_{1D} \cdot \frac{b}{As_{used}} = 28.27 \cdot \frac{1000}{280} = 100.98\ mm$$

$$s_{used} = 100\ mm$$

Check:

a. $s_{used} \leq 5h$

$100\ mm\ \leq 5 \cdot 140\ mm$

$100\ mm\ \leq 700\ mm$ (OK)

b. $s_{used} \leq 450\ mm$

$100\ mm\ \leq 450\ mm$ (OK)

Hence, it can be concluded that based on the calculations, the shear reinforcement (stirrup) used for the support area of floor plate in the X direction is P6-100 mm.

After determining the flexural and shear reinforcements used in the support area of floor plate in the X direction, the reinforcement design is also carried out for the middle span area in the X direction, as well as the support and middle span areas in the Y direction. The results are compiled in the following table.

Table 5.79 Floor Plate Reinforcements

Area	X Direction		Y Direction	
	Flexural	Shear	Flexural	Shear
Support	D10-200	P6-100	D10-200	P6-100
Middle Span	D10-200	P6-100	D10-200	P6-100

Finally, from the table above, it is concluded that the roof plate flexural reinforcements in both support and middle span areas are D10-200 mm, while the shear reinforcements are P6-100 mm.

5.6.5 Roof Plate Reinforcement Design

The initial data of the roof plate material properties is as follows.

f’c = 35 MPa

fy2 = 360 MPa

fy1 = 400 MPa

β = 0.80

$D_{flexural}$ = 10 mm

D_{shear} = 6 mm

h_{plate} = 0.10 m = 100 mm

Sb = 20 mm

ds = Sb + 0.5($D_{flexural}$) = 20 + 0.5(10) = 25 mm

Ly = 3 m = 3000 mm

Lx = 4 m = 4000 mm

Lny = 2650 mm

Lnx = 3600 mm

From the data above, the shortest side net length Ln is obtained at Lnx, which is 2650 mm or 2.65 m.

The dead load excluding the additional load is calculated as follows.

$$\text{Dead load} = \text{h}_{\text{plate}} \times \gamma_{\text{concrete}} = 0.10 \times 2400 = 240\ \text{kg/m}^2$$

Meanwhile, the additional dead load of the roof plate can be seen in the following table.

Table 5.80 Additional Dead Load of Roof Plate

No	Component	Volume Weight		Thickness		Q	
		Value	Unit	Value	Unit	kg/m²	kN/m²
1	Spec	21	kg/m²/cm thickness	3	cm	63	0.618
2	Mechanical & Electrical					30	0.294
3	Ceiling					9	0.088
4	Ceiling Hanger					5	0.049
5	Waterproofing	2100	kg/m³	0.02	m	42	0.412
Total Additional Dead Load						149	1.462

Hence, the ultimate load calculation of the roof plate is as follows.

$$\text{Qd} = \text{Dead load} + \text{Additional dead load} = 240 + 149 = 492.93\ \text{kg/m}^2$$

$$\text{Ql} = 2.40\ \text{kN/m}^2 = 244.65\ \text{kg/m}^2$$

$$\text{Qu} = 1.2 \cdot \text{Qd} + 1.6 \cdot \text{Ql} = 1.2 \cdot 492.93 + 1.6 \cdot 244.65$$

$$= 982.95\ \text{kg/m}^2 = 9.64\ \text{kN/m}^2$$

The coefficient parameters used to calculate the moment values according to the table from PBI 1971 are as follows.

Lx/Ly = 4000/3000 = 1.333 ≈ 1.3

x_{Mtx-} = 69

x_{Mlx+} = 31

x_{Mty-} = 57

x_{Mly+} = 19

Lx in this case is the shortest side, while Ly is the longest. So, for the calculation of the moments, Lx is 3 m while Ly is 4 m. Hence, the calculation of the moments is as follows.

$$Mtx^{-} = -0.001 \cdot Qu \cdot Lx^{2} \cdot x_{Mtx-} = -0.001 \cdot 9.64 \cdot 3^{2} \cdot 69 = -5.99 \text{ kNm}$$

$$Mlx^{+} = 0.001 \cdot Qu \cdot Lx^{2} \cdot x_{Mlx+} = 0.001 \cdot 9.64 \cdot 3^{2} \cdot 31 = 2.69 \text{ kNm}$$

$$Mty^{-} = -0.001 \cdot Qu \cdot Lx^{2} \cdot x_{Mty-} = -0.001 \cdot 9.64 \cdot 3^{2} \cdot 57 = -4.95 \text{ kNm}$$

$$Mly^{+} = 0.001 \cdot Qu \cdot Lx^{2} \cdot x_{Mly+} = 0.001 \cdot 9.64 \cdot 3^{2} \cdot 19 = 1.65 \text{ kNm}$$

Furthermore, the reinforcement design is divided into four types for one roof plate, which are the support and middle span areas in the X direction, as well as the support and middle span areas in the Y direction. The following calculation shows the reinforcement design of the support area in the X direction.

1. Shear force calculation

   $Vu = 0.5 \cdot 1.15 \cdot Qu \cdot Ln = 0.5 \cdot 1.15 \cdot 9.64 \cdot 2.65 = 14.69 \text{ kN}$

   $d = h - ds = 100 - 25 = 75 \text{ mm}$

   $b_{w} = 1000 \text{ mm}$

   $Vn = 0.17 \cdot \sqrt{f'c} \cdot b_{w} \cdot d = 0.17 \cdot \sqrt{35} \cdot 1000 \cdot 75 = 75430.02 \text{ N}$

   $\varphi = 0.75$

   $\varphi Vn = 0.75 \cdot 75430.02 = 56572.51 \text{ N} = 56.57 \text{ kN}$

   Check:

   $\varphi Vn > Vu$

   $56.57 \text{ kN} > 14.69 \text{ kN}$ (SAFE, plate dimension can withstand shear force)

2. Moment calculation

   $Mu = 5.99 \text{ kNm}$

   $\varphi = 0.90$

   $Mn = \frac{Mu}{\varphi} = \frac{5.99}{0.90} = 6.65 \text{ kNm}$

$b = 1000 \text{ mm}$

$d = 75 \text{ mm}$

Using quadratic formula to determine the value of a

$Mn = 0.85 \cdot f'c \cdot a \cdot b \cdot \left(d - \frac{a}{2}\right)$

$6.65 = 0.85 \cdot 35 \cdot a \cdot 1000 \cdot \left(75 - \frac{a}{2}\right)$

From the quadratic formula, the value of a is obtained 3.04 mm.

3. Flexural reinforcement design

The As area is determined with the Ts = Cc formula.

$As_{min} = 0.002 \cdot Ag = 0.002 \cdot (1000 \cdot 75) = 150 \text{ mm}^2$

$As_{needed} = 0.85 \cdot f'c \cdot a \cdot \frac{b}{fy} = 0.85 \cdot 35 \cdot 3.04 \cdot \frac{1000}{360} = 251.53 \text{ mm}^2$

$As_{balance} = 0.85 \cdot \beta_1 \cdot \frac{f'c}{fy} \cdot \frac{600}{600+} \cdot b \cdot d$

$As_{balance} = 0.85 \cdot 0.80 \cdot \frac{35}{360} \cdot \frac{600}{600+3} \cdot 1000 \cdot 75 = 3098.96 \text{ mm}^2$

$As_{max} = 0.75 \cdot As_{balance} = 0.75 \cdot 3098.96 = 2324.22 \text{ mm}^2$

$As_{used} = 251.53 \text{ mm}^2$

$As_{1D} = \frac{1}{4} \cdot \pi \cdot D_{flexural}{}^2 = \frac{1}{4} \cdot \pi \cdot 10^2 = 78.54 \text{ mm}^2$

$s = As_{1D} \cdot \frac{b}{As_{used}} = 78.54 \cdot \frac{1000}{251.53} = 312.25 \text{ mm}$

$s_{used} = 150 \text{ mm}$

$As_{reinforcement\ used} = \frac{b}{s_{used}} \cdot As_{1D} = \frac{1000}{150} \cdot 78.54 = 523.60 \text{ mm}^2$

Check:

a. $s_{used} \leq 2d$

$150 \text{ mm} \leq 2 \cdot 75 \text{ mm}$

$150 \text{ mm} \leq 150 \text{ mm}$ (OK)

b. $s_{used} \leq 450 \text{ mm}$

$150 \text{ mm} \leq 450 \text{ mm}$ (OK)

Hence, it can be concluded that based on the calculations, the flexural reinforcement used for the support area of roof plate in the X direction is D10-150 mm.

4. Shear reinforcement design

$\mathrm{As}_{\mathrm{shear}} = 0.002 \cdot \mathrm{b} \cdot \mathrm{h} = 0.002 \cdot 1000 \cdot 100 = 200\ \mathrm{mm}^2$

$\mathrm{As}_{1\mathrm{D}} = \frac{1}{4} \cdot \pi \cdot \mathrm{D}_{\mathrm{shear}}{}^2 = \frac{1}{4} \cdot \pi \cdot 6^2 = 28.27\ \mathrm{mm}^2$

$\mathrm{s} = \mathrm{As}_{1\mathrm{D}} \cdot \frac{\mathrm{b}}{\mathrm{As}_{\mathrm{used}}} = 28.27 \cdot \frac{1000}{200} = 141.37\ \mathrm{mm}$

$\mathrm{s}_{\mathrm{used}} = 100\ \mathrm{mm}$

Check:

a. $\mathrm{s}_{\mathrm{used}} \leq 5\mathrm{h}$

$100\ \mathrm{mm}\ \leq 5 \cdot 100\ \mathrm{mm}$

$100\ \mathrm{mm}\ \leq 500\ \mathrm{mm}$ (OK)

b. $\mathrm{s}_{\mathrm{used}} \leq 450\ \mathrm{mm}$

$100\ \mathrm{mm}\ \leq 450\ \mathrm{mm}$ (OK)

Hence, it can be concluded that based on the calculations, the shear reinforcement (stirrup) used for the support area of roof plate in the X direction is P6-100 mm.

After determining the flexural and shear reinforcements used in the support area of floor plate in the X direction, the reinforcement design is also carried out for the middle span area in the X direction, as well as the support and middle span areas in the Y direction. The results are compiled in the following table.

Table 5.81 Roof Plate Reinforcements

Area	X Direction		Y Direction	
	Flexural	Shear	Flexural	Shear
Support	D10-150	P6-100	D10-150	P6-100
Middle Span	D10-150	P6-100	D10-150	P6-100

Finally, from the table above, it is concluded that the roof plate flexural reinforcements in both support and middle span areas are D10-150 mm, while the shear reinforcements are P6-100 mm.

5.6.6 Stairs Reinforcement Design

The ultimate moments of the stairs and stair landing obtained from the ETABS model analysis is as follows.

Stairs: $Mu^{-} = 77.14$ kN-m

$Mu^{+} = 146.09$ kN-m

Stair landing: $Mu^{-} = 77.14$ kN-m

$Mu^{+} = 146.09$ kN-m

The flexural reinforcement design for the stairs in the support area is as follows.

$\varphi = 0.9$

$Mn^{-} = \frac{Mu^{-}}{\varphi} = \frac{77.14}{0.9} = 85.71 \text{ kN-m}$

$\text{Concrete cover (Sb)} = 40 \text{ mm}$

$D_{flexural} = 25 \text{ mm}$

$D_{shear} = 10 \text{ mm}$

$ds = Sb + \frac{D_{flexural}}{2} = 52.5 \text{ mm}$

$H = t = 300 \text{ mm}$

$d = H - ds = 247.5 \text{ mm}$

$B = 1.45 \text{ m} = 1450 \text{ mm}$

To find the value of a, a quadratic formula is used as follows.

$Mn = Cc\left(d - \frac{a}{2}\right)$

$85714222.22 = 7363125a - 14875a^2$

$14875a^2 - 7363125a + 85714222.22 = 0$

From the formula, the value of a is obtained 11.93 mm. Furthermore, the value of As is determined.

$As = \frac{0.85 \cdot f'c \cdot a \cdot B}{fy} = 880.32 \text{ mm}^2$

Meanwhile, the reinforcement ratio (ρ) is as follows.

$\rho_{min} = \frac{1.4}{fy} = 0.0035$

$\rho = \frac{As}{B \cdot d} = 0.0025$

The value of p used is the largest one, hence the reinforcement ratio used is 0.0035. Meanwhile, the value of As used is as follows.

$As_{used} = \rho_{used} \cdot B \cdot d = 1256.06 \text{ mm}^2$

The flexural reinforcement spacing (s) is determined as follows.

$$\mathrm{s} = \frac{\frac{1}{4}\cdot\pi\cdot \mathrm{D_{flexural}}^2\cdot \mathrm{B}}{\mathrm{As_{used}}} = 566.67\ \mathrm{mm}$$

After further consideration, the spacing (s) value used for flexural reinforcement of the stairs in the support area is 150 mm. Hence, the identity is D25-150 mm. Moreover, the shear reinforcement is also determined as follows.

$$\rho_{\mathrm{shear}} = 0.002$$

$$\mathrm{As_{used}} = \rho_{\mathrm{shear}} \cdot \mathrm{B} \cdot \mathrm{d} = 717.75\ \mathrm{mm}^2$$

$$\mathrm{s} = \frac{\frac{1}{4}\cdot\pi\cdot \mathrm{D_{shear}}^2\cdot \mathrm{B}}{\mathrm{As_{used}}} = 158.67\ \mathrm{mm}$$

After further consideration, the spacing (s) value used for shear reinforcement of the stairs in the support area is 150 mm. Hence, the identity is P10-150 mm.

The reinforcement design for the stairs in the middle span area is then carried out as well, which can be seen as follows.

$$\varphi = 0.9$$

$$\mathrm{Mn}^+ = \frac{\mathrm{Mu}^+}{\varphi} = \frac{146.09}{0.9} = 162.32\ \mathrm{kN\text{-}m}$$

$$\mathrm{Concrete\ cover\ (Sb)} = 40\ \mathrm{mm}$$

$$\mathrm{D_{flexural}} = 25\ \mathrm{mm}$$

$$\mathrm{D_{shear}} = 10\ \mathrm{mm}$$

$$\mathrm{ds} = \mathrm{Sb} + \frac{\mathrm{D_{flexural}}}{2} = 52.5\ \mathrm{mm}$$

$$\mathrm{H} = \mathrm{t} = 300\ \mathrm{mm}$$

$$\mathrm{d} = \mathrm{H} - \mathrm{ds} = 247.5\ \mathrm{mm}$$

$$\mathrm{B} = 1.45\ \mathrm{m} = 1450\ \mathrm{mm}$$

To find the value of a, a quadratic formula is used as follows.

$$\mathrm{Mn} = \mathrm{Cc}\left(\mathrm{d} - \frac{\mathrm{a}}{2}\right)$$

$$162317333.33 = 10676531\mathrm{a} - 21569\mathrm{a}^2$$

$$21569\mathrm{a}^2 - 10676531\mathrm{a} + 162317333.33 = 0$$

From the formula, the value of a is obtained 15.70 mm. Furthermore, the value of As is determined.

$$\mathrm{As} = \frac{0.85\cdot f'c\cdot \mathrm{a}\cdot \mathrm{B}}{\mathrm{fy}} = 1693.28\ \mathrm{mm}^2$$

Meanwhile, the reinforcement ratio (ρ) is as follows.

$$\rho_{\text{min}} = \frac{1.4}{\text{fy}} = 0.0035$$

$$\rho = \frac{\text{As}}{\text{B}\cdot\text{d}} = 0.0047$$

The value of ρ used is the largest one, hence the reinforcement ratio used is 0.0047. Meanwhile, the value of As used is as follows.

$$\text{As}_{\text{used}} = \rho_{\text{used}} \cdot \text{B} \cdot \text{d} = 1693.28\ \text{mm}^2$$

The flexural reinforcement spacing (s) is determined as follows.

$$\text{s} = \frac{\frac{1}{4}\cdot\pi\cdot{\text{D}_{\text{flexural}}}^2\cdot\text{B}}{\text{As}_{\text{used}}} = 420.35\ \text{mm}$$

After further consideration, the spacing (s) value used for flexural reinforcement of the stairs in the support area is 150 mm. Hence, the identity is D25-150 mm. Moreover, the shear reinforcement is also determined as follows.

$$\text{p}_{\text{shear}} = 0.002$$

$$\text{As}_{\text{used}} = \rho_{\text{shear}} \cdot \text{B} \cdot \text{d} = 717.75\ \text{mm}^2$$

$$\text{s} = \frac{\frac{1}{4}\cdot\pi\cdot{\text{D}_{\text{shear}}}^2\cdot\text{B}}{\text{As}_{\text{used}}} = 158.67\ \text{mm}$$

After further consideration, the spacing (s) value used for shear reinforcement of the stairs in the support area is 150 mm. Hence, the identity is P10-150 mm.

As the reinforcement design for the stairs is finished, the reinforcement for the stair landing is also carried out with the same design process. The conclusion of the results of the reinforcement used for the whole stairs element can be seen in the following table.

Table 5.82 Reinforcement of the Stairs Element

Element	Support Area		Middle Span Area	
	Flexural	Shear	Flexural	Shear
Stairs	D25-150	P10-150	D25-150	P10-150
Stair landing	D25-150	P10-150	D25-150	P10-150

Finally, from the table above, it is concluded that the stairs and stair landing flexural reinforcements in both support and middle span areas are D25-150 mm, while the shear reinforcements are P10-150 mm.

5.7 Foundation Design

The foundation used in this design is the pile foundation type, which will be designed in a group with a pile cap. The initial properties assumption of the pile considered for the design are as follows.

Pile type = Circular bored pile

Pile diameter (D) = 0.85 m = 85 cm = 2.79 ft

Pile length (L) = 15 m = 1500 cm = 49.22 ft

Pile end area (Ap) = $(1/4)\ \pi \times D^2 = (1/4)\ \pi \times 0.85^2 = 0.57\ m^2 = 6.11\ ft^2$

Cover area (As) = $\pi \times D \times L = \pi \times 0.85 \times 15 = 40.06\ m^2 = 431.17\ ft^2$

f'c of concrete = 35 MPa = 356.90 kg/cm^2 ≈ 357 kg/cm^2

With the specific gravity of concrete of 2400 kg/m^3, the pile weight is determined as follows.

Pile weight (Wp) = $(1/4)\ \pi \times D^2 \times L \times 2400 = (1/4)\ \pi \times 0.85^2 \times 15 \times 2400$

= 20428.21 kg = 20.43 ton

5.6.1 Standard Penetration Test (N-SPT) Data

The N-SPT data is obtained by Universitas Islam Indonesia (UII) in the location of Pleret, Imogiri, Bantul, Yogyakarta. The soil data of N-SPT obtained at each depth is as follows.

Table 5.83 N-SPT Data

Depth	N-SPT
0	0
2	18
4	14
6	24
8	24
10	12
12	18
14	37
16	60
18	45
20	60

This data is used to design the pile foundation along with its pile cap. The average value of the N-SPT data along the pile length is 25.88. Because the average value is more than 15, the soil is classified as medium soil. Meanwhile, as the pile length is 15 m, the N-SPT data at the end of the pile length is taken as 60 (according to 16 m depth data).

5.6.2 Bearing Capacity Analysis

The P, Mx, and My values according to workload, factored gravity load and factored earthquake load working on the base of the building obtained from the ETABS model analysis can be seen in the following table.

Table 5.84 P, Mx, and My Values for Pile Foundation Design

Condition (Load Combination)	P (ton)	Mx (ton-m)	My (ton-m)
Workload: 1D + 1L	1087.83	11.34	8.00
Factored gravity load: 1.2D + 1.6 L	1391.02	14.89	10.58
Factored earthquake load: 1.2D + 0.5L + 1E	1049.08	151.26	126.59

According to Meyerhof's method for bearing capacity shown in Equations 3.141 to 3.143, the calculation of ultimate bearing capacity is as follows.

End bearing capacity (Qp) $= 40 \text{ ton/m}^2 \times Ap \times N_{\text{pile end}}$

$= 40 \times 0.57 \times 60 = 1361.88$ ton

Cover bearing capacity (Qs) $= 0.2 \text{ ton/m}^2 \times As \times N_{\text{average}}$

$= 0.2 \times 40.06 \times 25.88 = 207.29$ ton

Ultimate bearing capacity (Qu) $= Qp + Qs$

$= 1361.88 + 207.29 = 1569.17$ ton

The Safety Factor (SF) requirement is between 2.5 and 4. In this analysis, the SF value of 4 is used. Hence, the allowable bearing capacity is as follows.

Allowable bearing capacity (Q_{all}) $= Qu/SF$

$= 1569.17/4 = 392.29$ ton

5.6.3 Dimension Estimation

A trial-and-error process with an initial assumption of the dimension is needed to design the pile cap. The following calculation according to Equation 3.145 shows the initial assumption of the number of piles.

Number of piles in a group (n_{pile}) $= P_{\text{max}}/Q_{\text{all}} = 1391.02/392.29 = 3.55$

The number of piles needed in a group is obtained as 3.55 or rounded up to 4 piles. However, after further consideration, the number of piles used is 6.

Number of piles used (n_{used}) $= 6$

Number of rows (m) = 2

Number of piles in a row (n) = 3

As the spacing requirement according to Equation 3.146 is 2.5D ≤ S ≤ 3D, the spacing calculation uses the requirement of 3D.

Spacing (s) = 3 × D = 3 × 0.85 = 2.55 m

The spacing between the outermost pile and the pile cap edge is taken as equal to the diameter (D) of the pile, which is 0.85 m. Therefore, the spacing between the center of the outermost pile and the pile cap edge is 1.5D.

Spacing of the edge = 1.5 × D = 1.5 × 0.85 = 1.275 m

The following figure shows the dimensions of the pile cap.

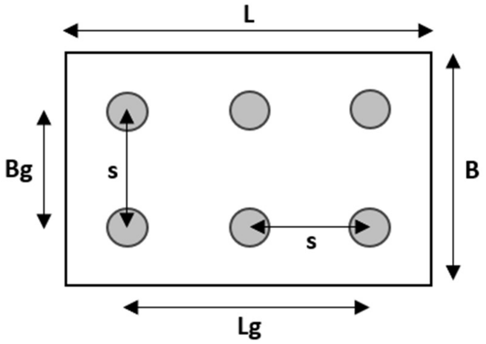


Figure 5.21 Pile Cap Dimension

Meanwhile, the length and the width of the pile group are as follows.

Group length (Lg) = 2 × s = 2 × 2.55 = 5.10 m

Group width (Bg) = 1 × s = 1 × 2.55 = 2.55 m

The efficiency of pile group is calculated according to Equations 3.147 and 3.148 as follows.

$$\theta = \arctan(D/s) = \arctan(0.85/2.55) = \arctan 0.33 = 18.43°$$

$$Eg = 1 - \frac{\theta}{90} \times \left(\frac{(n-1)m + (m-1)n}{mn}\right) = 1 - \frac{18.43}{90} \times \left(\frac{(3-1)2 + (2-1)3}{2 \cdot 3}\right) = 0.76$$

With 6 total piles in a group, the total bearing capacity of individual piles considering the group efficiency is calculated according to Equation 3.149 as follows.

Total bearing capacity (ΣQ_{all}) = Eg × n × Q_{all} = 0.76 × 6 × 392.29 = 1791.27 ton

To check if the total bearing capacity fits according to the requirements, the value of ΣQ_{all} must be larger than the maximum P load value obtained with different load combinations from the ETABS analysis. The largest is the factored gravity load of 1391.02 ton. Because the value of 1791.27 ton is larger than 1391.02 ton, the total bearing capacity is considered to have fit the requirements.

As the group efficiency is considered appropriate, the calculation of the pile cap dimensions (length and width) are as follows.

Pile cap length (L) $= Lg + 2(1.5 \times D) = 5.10 + 2(1.275) = 7.65$ m

Pile cap width (B) $= Bg + 2(1.5 \times D) = 2.55 + 2(1.275) = 5.10$ m

Meanwhile, the thickness (t) of the pile cap is assumed to be 2.5 m. Furthermore, with the concrete volume weight ($\gamma_{concrete}$) of 2.4 ton/m^3, the weight of the pile cap ($W_{pile\ cap}$) according to Equation 3.150 is calculated as follows.

$$W_{\text{pile cap}} = \gamma_{\text{concrete}} \times L \times B \times t = 2.4 \times 7.65 \times 5.10 \times 2.5 = 234.09 \text{ ton}$$

The maximum and minimum axial force of a pile group are then determined to check the fulfillment of the requirement where the ultimate bearing capacity of each condition (workload, factored gravity load and factored earthquake load) has a higher value than both maximum and minimum axial forces. To determine the value of the axial forces, the maximum arm of the pile in the x direction (x_{max}) and y direction (y_{max}) to the center of gravity are calculated. The calculation is as follows.

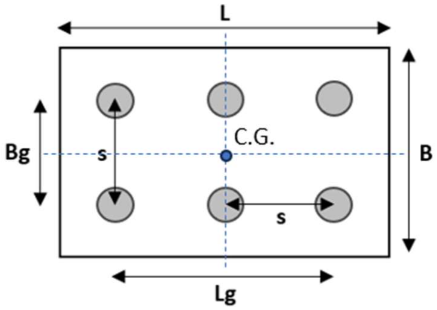


Figure 5.22 Arm Length of Piles to the Center of Gravity

x_{max} = 2.55 m

y_{max} = 1.275 m

To calculate the axial forces, the values of Σx^2 and Σy^2 are needed. The calculations are as follows.

Table 5.85 Calculation of Σx^2 of Pile Cap

No	Number of piles	x (m)	n.x²
1	2	2.55	13.01
2	2	-2.55	13.01
Σn	4	Σx^2	26.01

Table 5.86 Calculation of Σy^2 of Pile Cap

No	Number of piles	y (m)	n.y²
1	3	1.275	4.88
2	3	-1.275	4.88
Σn	6	Σy^2	9.75

The analysis of ultimate bearing capacity influenced by each condition can be seen as follows.

1. Workload (1D + 1L)

P = 1087.83 ton

$P_{\text{pile cap}}$ = 234.09 ton

Mx = 11.34 ton-m

My = 8.00 ton-m

The calculation according to Equation 3.151 of the maximum axial force with workload is as follows.

$$P_{max} = \left(\frac{P+P_{\text{pile cap}}}{n_{used}}\right) + \left(\frac{Mx \cdot x_{max}}{\Sigma x^2}\right) + \left(\frac{My \cdot y_{max}}{\Sigma y^2}\right)$$

$$P_{max} = \left(\frac{1087.83+234.09}{6}\right) + \left(\frac{11.34 \cdot 2.55}{26.01}\right) + \left(\frac{8.00 \cdot 1.275}{9.75}\right) = 222.48 \text{ ton}$$

The calculation according to Equation 3.152 of the minimum axial force with workload is as follows.

$$P_{min} = \left(\frac{P+P_{\text{pile cap}}}{n_{used}}\right) - \left(\frac{Mx \cdot x_{max}}{\Sigma x^2}\right) - \left(\frac{My \cdot y_{max}}{\Sigma y^2}\right)$$

$$P_{min} = \left(\frac{1087.83+234.09}{6}\right) - \left(\frac{11.34 \cdot 2.55}{26.01}\right) - \left(\frac{8.00 \cdot 1.275}{9.75}\right) = 218.16 \text{ ton}$$

Meanwhile, the allowable bearing capacity influenced by workload with the consideration of group efficiency is as follows.

Q_{all} workload = Q_{all} × Eg = 392.29 × 0.76 = 298.55 ton

Because 298.55 ton is a larger value than P_{max} and P_{min} of the workload condition, the allowable bearing capacity is considered safe.

2. Factored gravity load (1.2D + 1.6L)

   P = 1391.02 ton

   $P_{\text{pile cap}}$ = 234.09 ton

   Mx = 14.89 ton-m

   My = 10.58 ton-m

   The calculation according to Equation 3.151 of the maximum axial force with factored gravity load is as follows.

   $$P_{\max} = \left(\frac{P+P_{\text{pile cap}}}{n_{\text{used}}}\right) + \left(\frac{Mx \cdot x_{\max}}{\Sigma x^2}\right) + \left(\frac{My \cdot y_{\max}}{\Sigma y^2}\right)$$

   $$P_{\max} = \left(\frac{1391.02+2\quad .09}{6}\right) + \left(\frac{14.89 \cdot 2.55}{26.01}\right) + \left(\frac{10.58 \cdot 1.275}{9.75}\right) = 273.69 \text{ ton}$$

   The calculation according to Equation 3.152 of the minimum axial force with factored gravity load is as follows.

   $$P_{\min} = \left(\frac{P+P_{\text{pile cap}}}{n_{\text{used}}}\right) - \left(\frac{Mx \cdot x_{\max}}{\Sigma x^2}\right) - \left(\frac{My \cdot y_{\max}}{\Sigma y^2}\right)$$

   $$P_{\min} = \left(\frac{1391.02+234.09}{6}\right) - \left(\frac{14.89 \cdot 2.55}{26.01}\right) - \left(\frac{10.58 \cdot 1.275}{9.75}\right) = 268.01 \text{ ton}$$

   Meanwhile, the ultimate bearing capacity influenced by factored gravity load with the consideration of group efficiency is as follows.

   Qu workload = Qu × Eg = 1569.17 × 0.76 = 1194.18 ton

   Because 1194.18 ton is a larger value than $P_{\max}$ and $P_{\min}$ of the factored gravity load condition, the ultimate bearing capacity is considered safe.

3. Factored earthquake load (1.2D + 0.5L +1E)

   P = 1049.08 ton

   $P_{\text{pile cap}}$ = 234.09 ton

   Mx = 151.26 ton-m

   My = 126.59 ton-m

   The calculation according to Equation 3.151 of the maximum axial force with factored earthquake load is as follows.

   $$P_{\max} = \left(\frac{P+P_{\text{pile cap}}}{n_{\text{used}}}\right) + \left(\frac{Mx \cdot x_{\max}}{\Sigma x^2}\right) + \left(\frac{My \cdot y_{\max}}{\Sigma y^2}\right)$$

   $$P_{\max} = \left(\frac{1049.08+2\quad .09}{6}\right) + \left(\frac{151.26 \cdot 2.55}{26.01}\right) + \left(\frac{126.59 \cdot 1.275}{9.75}\right) = 245.24 \text{ ton}$$

The calculation according to Equation 3.152 of the minimum axial force with factored earthquake load is as follows.

$$P_{min} = \left(\frac{P+P_{pile\ cap}}{n_{used}}\right) - \left(\frac{Mx \cdot x_{max}}{\Sigma x^2}\right) - \left(\frac{My \cdot y_{max}}{\Sigma y^2}\right)$$

$$P_{min} = \left(\frac{1049.08+234.09}{6}\right) - \left(\frac{151.26 \cdot 2.55}{26.01}\right) - \left(\frac{126.59 \cdot 1.275}{9.75}\right) = 182.49 \text{ ton}$$

Meanwhile, the ultimate bearing capacity influenced by factored earthquake load with the consideration of group efficiency is as follows.

Qu workload = Qu × Eg = 1569.17 × 0.76 = 1194.18 ton

Because 1194.18 ton is a larger value than P_{max} and P_{min} of the factored earthquake load condition, the ultimate bearing capacity is considered safe.

As all the bearing capacity requirements in each condition are considered safe, the estimated dimensions of the pile cap that was previously analyzed can be used and are acceptable.

5.6.4 Reinforcement Design

Concluded from the dimension estimation of pile cap, the following data shows the dimensions of pile cap used.

Length (L) = 7.65 m

Width (B) = 5.10 m

Thickness (t) = 2.5 m = 250 cm

Meanwhile, the quality of the concrete (f'c) and steel (fy) are as follows.

f'c = 35 MPa = 357 kg/cm^2

fy = 400 MPa = 4080 kg/cm^2

The concrete cover thickness (Sb) is 75 mm or 7.5 cm. Furthermore, the initial assumption for the reinforcement of the pile cap is as follows.

$Ø_{flexural}$ = 32 mm = 3.2 cm

$Ø_{shear}$ = 19 mm = 1.9 cm

$A_{flexural} = (1/4)\ \pi \times D^2 = (1/4)\ \pi \times Ø_{flexural}^2 = 8.04 \text{ cm}^2$

$A_{shear} = (1/4)\ \pi \times D^2 = (1/4)\ \pi \times Ø_{shear}^2 = 2.84 \text{ cm}^2$

The maximum axial force (P_{max}) and minimum axial force (P_{min}) used in this analysis is the largest among the three conditions (workload, factored gravity load and factored earthquake load). Hence, the P_{max} value used is 312.95 ton and the P_{min} value used is 273.92 ton.

The additional data needed for the design of pile cap reinforcement can be seen as follows.

x_{max} = 2.55 m

y_{max} = 1.275 m

H_{column} = 1.2 m = 120 cm

B_{column} = 1.0 m = 100 cm

Meanwhile, the reduction factors for flexural and shear reinforcements are determined as follows.

$\phi_{flexural}$ = 0.80

ϕ_{shear} = 0.75

With the obtained data, the ultimate and nominal moments of the pile cap are determined using Equations 3.153 and 3.154 as follows.

$$Mu = \frac{n \cdot P_{max} \cdot (x_{max} - 0.5 \cdot B_{column})}{L} = \frac{3 \cdot 312.95 \cdot (2.55 - 0.5 \cdot 1)}{7.65} = 251.59 \text{ ton-m}$$

$$Mn = \frac{Mu}{\phi_{flexural}} = \frac{251.59}{0.80} = 314.49 \text{ ton-m}$$

Furthermore, the effective height (thickness) of the pile cap is determined with Equation 3.155 as follows.

$$h = t - Sb - \frac{\phi_{flexural}}{2} = 250 - 7.5 - \frac{3.2}{2} = 240.90 \text{ cm}$$

Meanwhile, the pile cap is checked to be able to withstand the shear forces. The control is checked towards one-way and two-way shears, which can be seen as follows.

1. Pile cap control towards one-way shear

   Spacing between piles (s) = 2.55 m = 255 cm

   The shear plane is determined with Equation 3.156 as follows.

   Shear plane = H_{column}/2 + B = 120/2 + 510
   = 570 cm

   According to the requirement, the shear plane must be longer than the spacing between piles (s). Because 570 cm is longer than 255 cm, the pile cap is considered safe or fulfills the requirement.

2. Pile cap control towards two-way shear

The calculations according to Equations 3.157 to 3.162 for pile cap control towards two-way shear are as follows.

P_{max} = 273.69 ton

P_{min} = 268.01 ton

$$Vu = n(P_{max} + P_{min}) = 3(273.69 + 268.01) = 1625.11 \text{ ton}$$

$$bo = n(H_{column} + B) = 3(120 + 510) = 1890 \text{ cm}$$

$$\beta_{column} = \frac{B_{column}}{H_{column}} = \frac{100}{120} = 0.83$$

$$\phi Vc1 = \left(\phi\left(1 + \frac{2}{\beta_{column}}\right)\frac{\sqrt{f'c}\cdot bo\cdot h}{6}\right)$$

$$= \left(0.75\left(1 + \frac{2}{0.83}\right)\frac{\sqrt{35}\cdot 18900\cdot 2409}{6}\right)$$

$$= 114477874 \text{ N} = 11669.51 \text{ ton}$$

$$\phi Vc2 = \left(\phi\left(2 + \frac{\alpha_s\cdot h}{bo}\right)\frac{\sqrt{f'c}\cdot bo\cdot h}{12}\right)$$

$$= \left(0.75\left(2 + \frac{40\cdot 2409}{18900}\right)\frac{\sqrt{35}\cdot 18900\cdot 2409}{12}\right)$$

$$= 119501647 \text{ N} = 12181.62 \text{ ton}$$

$$\phi Vc3 = \left(\phi\left(\frac{1}{3}\right)\sqrt{f'c}\cdot bo\cdot h\right)$$

$$= \left(0.75\left(\frac{1}{3}\right)\sqrt{35}\cdot 18900\cdot 2409\right)$$

$$= 67339926 \text{ N} = 6864.42 \text{ ton}$$

Among the ϕVc values, the minimum value is 6864.42 ton. Because ϕVc_{min} is still bigger than Vu, the pile cap is considered safe or that it fulfills the requirement.

As the data needed to analyze the reinforcement of the foundation are obtained, the flexural and shear reinforcement design are as follows.

1. Pile cap flexural reinforcement design

The minimum reinforcement ratio (ρ_{min}) for the flexural reinforcement design is determined with Equation 3.163 as follows.

$$\rho_{min} = \frac{1.4}{fy} = \frac{1.4}{400} = 0.0035 = 0.35\%$$

The design is carried out for both the X and Y directions of the pile cap. The ultimate cross-sectional area of the flexural reinforcement in the X and Y directions of the pile cap is as follows.

Bx = Lx = 765 cm

By = Ly = 510 cm

The required area of flexural reinforcement is determined with Equation 3.164 as follows.

$$\mathrm{As}_{\text{flexural of X direction}} = \rho_{\min} \cdot \mathrm{B_x} \cdot \mathrm{h}$$
$$= 0.0035 \cdot 765 \cdot 240.90 = 645.01\ \mathrm{cm^2}$$

$$\mathrm{As}_{\text{flexural of Y direction}} = \rho_{\min} \cdot \mathrm{B_y} \cdot \mathrm{h}$$
$$= 0.0035 \cdot 510 \cdot 240.90 = 430.01\ \mathrm{cm^2}$$

Finally, the spacing of flexural reinforcement is determined with Equation 3.165 as follows.

$$\mathrm{s}_{\text{X direction}} = \frac{\mathrm{B_x} \cdot \mathrm{A}_{\text{flexural}}}{\mathrm{As}_{\text{flexural of X direction}}} = \frac{765 \times 8.04}{645.01} = 9.54\ \mathrm{cm}$$

$$\mathrm{s}_{\text{Y direction}} = \frac{\mathrm{B_y} \cdot \mathrm{A}_{\text{flexural}}}{\mathrm{As}_{\text{flexural of Y direction}}} = \frac{510 \times 8.04}{430.01} = 9.54\ \mathrm{cm}$$

After further consideration, the spacing used for flexural reinforcement is 5 cm or 50 mm for both the X and Y directions. Hence, the identity of the flexural reinforcement is D32-50 mm.

2. Pile cap shear reinforcement design

The minimum reinforcement ratio ($\rho_{\min}$) for the shear reinforcement design is used 0.0018 or 0.18%.

The design is carried out for both the X and Y directions of the pile cap. The ultimate cross-sectional area of the shear reinforcement in the X and Y directions of the pile cap is as follows.

Bx = Lx = 765 cm

By = Ly = 510 cm

The required area of shear reinforcement is determined with Equation 3.164 as follows.

$$\mathrm{As}_{\text{flexural of X direction}} = \rho_{\min} \cdot \mathrm{B_x} \cdot \mathrm{h}$$
$$= 0.0018 \cdot 765 \cdot 240.90 = 331.72\ \mathrm{cm^2}$$

$$\mathrm{As}_{\text{flexural of Y direction}} = \rho_{\text{min}} \cdot \mathrm{B_y} \cdot \mathrm{h}$$
$$= 0.0018 \cdot 510 \cdot 240.90 = 221.15 \text{ cm}^2$$

Finally, the spacing of shear reinforcement is determined with Equation 3.165 as follows.

$$\mathrm{s}_{\text{X direction}} = \frac{\mathrm{B_x} \cdot \mathrm{A_{shear}}}{\mathrm{As}_{\text{shear of X direction}}} = \frac{765 \times 2.84}{645.01} = 6.54 \text{ cm}$$

$$\mathrm{s}_{\text{Y direction}} = \frac{\mathrm{B_y} \cdot \mathrm{A_{shear}}}{\mathrm{As}_{\text{shear of Y direction}}} = \frac{510 \times 2.84}{430.01} = 6.54 \text{ cm}$$

After further consideration, the spacing used for shear reinforcement is 5 cm or 50 mm for both the X and Y directions. Hence, the identity of the shear reinforcement is D19-50 mm.

3. Bored pile flexural reinforcement design

   The minimum reinforcement ratio (ρ_{min}) for the flexural reinforcement design is used 0.015 or 1.5%. The required area of flexural reinforcement is determined with Equation 3.166 as follows.

   $$\mathrm{As}_{\text{flexural}} = \rho_{\text{min}} \cdot \mathrm{Ap} = 0.015 \cdot (0.57 \cdot 10^4) = 85.12 \text{ cm}^2$$

   The minimum number of flexural reinforcements needed in the bored pile is determined with Equation 3.167 as follows.

   $$\mathrm{n}_{\text{min}} = \frac{\mathrm{As}_{\text{flexural}}}{\mathrm{A}_{\text{flexural}}} = \frac{85.12}{8.04} = 10.58$$

   After further consideration, the number of flexural reinforcements used is 12. Hence, the identity of the flexural reinforcement is 12D32.

4. Bored pile shear reinforcement design

   The bored pile shear reinforcement requires a certain value of spacing (s) to not surpass the value of the requirements, which are analyzed as follows.

   a. h/2 = 240.90/2 = 120.45 cm
   b. $16D_{\text{flexural}}$ = 16(3.2) = 51.20 cm
   c. $48D_{\text{shear}}$ = 48(1.9) = 91.20 cm
   d. Minimum length of pile cap dimension = 510 cm

   The maximum spacing value allowed to be used is the smallest value among the requirements above, therefore the allowable spacing value is 51.20 cm. After further consideration, the spacing used for shear reinforcement in the first fifth of the pile length, which in this case is 3 m in length, is 5 cm or 50 mm. Hence, the identity of the shear reinforcement in this area is D19-50 mm. Meanwhile, the spacing used for shear reinforcement in the rest of the pile length is 10 cm or 100 mm. Hence, the identity of the shear reinforcement in this area is D19-100 mm.

5.8 Flexible Foundation

The flexible foundation includes springs with stiffness values that must be analyzed beforehand. The data for the flexible support design of the pile foundation is as follows.

W_{pile} = 20.43 ton

L_{pile} = 15 m = 49.21 ft

D_{pile} = 0.85 m = 2.79 ft

r_0 = 1.39 ft = 16.73 in

EI = 1.2×10^{10} lb-in^2

Gs = 400 t/ft^2

g = 9.81 m/s^2 = 32.2 ft/s^2

γ_{sat} = 110 lb/ft^3

γ_{pile} = 150 lb/ft^3

E_{pile} = 250000 t/ft^2

v_{soil} = 0.5

A_{pile} = 0.567 m^2 = 6.108 ft^2

Meanwhile, the data of pile cap dimensions are as follows.

$L_{pile\ cap}$ = 7.65 m = 25.10 ft

$B_{pile\ cap}$ = 5.10 m = 16.73 ft

$t_{pile\ cap}$ = 2.50 m = 8.20 ft

With the initial data obtained, the following figure shows the illustration of springs and dashpots in flexible foundation—whereas in this study, only springs are considered.

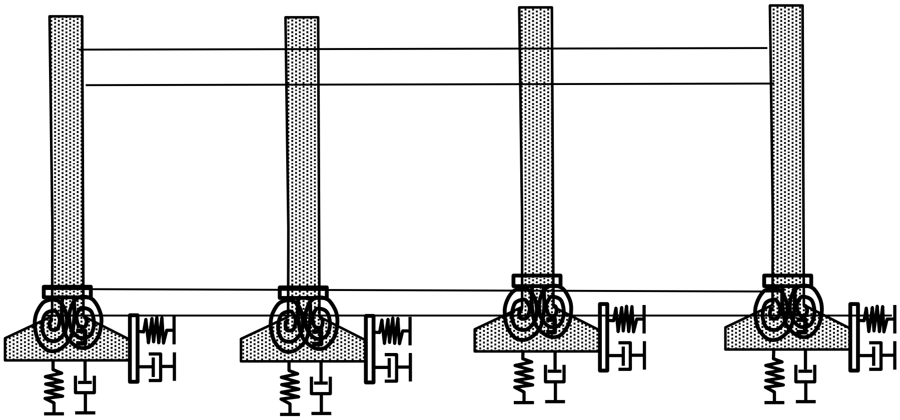


Figure 5.23 Illustration of Springs and Dashpots in Flexible Foundation

The figure above shows the location of springs and dashpots working in a flexible foundation beneath a column. Meanwhile, the information regarding each vibration type of the springs and dashpots is shown in the following figure.

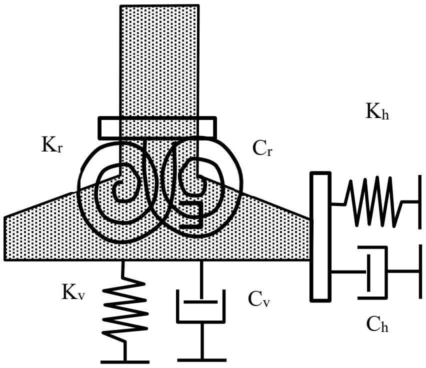


Figure 5.24 Spring and Dashpot Names According to Vibration Type

In the figure above, the names of spring stiffness and dashpot damping are mentioned based on the vibration type. The meaning of each name is as follows.

Kv = Spring stiffness according to vertical vibration

Kh = Spring stiffness according to horizontal vibration

Kr = Spring stiffness according to rocking vibration

Cv = Dashpot damping according to vertical vibration

Ch = Dashpot damping according to horizontal vibration

Cr = Dashpot damping according to rocking vibration

Each vibration type of spring stiffness is then analysed further in the following subchapters.

5.7.1 Vertical Vibration

According to Figure 3.18, the stiffness factor for a fixed tip vertically vibrating pile (f_{w1}) of the homogeneous soil profile graph is determined as follows.

Ep/Gs = 250000/400 = 625

L/r_0 = 49.21/1.39 = 35.29

Ep/Gs = 500 $f_{w1} = 0.041$

Ep/Gs = 1000 $f_{w1} = 0.029$

The values are then interpolated to determine the f_{w1} value with Ep/Gs equal to 625. The interpolation result is as follows.

Ep/Gs = 625 $f_{w1} = 0.038$

Meanwhile, the stiffness factor for a fixed tip vertically vibrating pile (f_{w1}) of the parabolic soil profile graph is determined as follows.

Ep/Gs = 250000/400 = 625

L/r_0 = 49.21/1.39 = 35.29

Ep/Gs = 500 $f_{w1} = 0.034$

Ep/Gs = 1000 $f_{w1} = 0.030$

The values are then interpolated to determine the f_{w1} value with Ep/Gs equal to 625. The interpolation result is as follows.

Ep/Gs = 625 $f_{w1} = 0.033$

The spring stiffness constant k_w of one pile in a vertical direction for the homogeneous soil profile according to Equation 3.168 is calculated as follows.

$$k_w^1 = \frac{E_p \cdot A}{r_0} f_w^1 = \frac{250000 \cdot 6.108}{1.39} 0.038 = 41615 \text{ t/ft} = 3468 \text{ t/in}$$

The spring stiffness constant k_w of one pile in a vertical direction for the parabolic soil profile is as follows.

$$k_w^1 = \frac{E_p \cdot A}{r_0} f_w^1 = \frac{250000 \cdot 6.108}{1.39} 0.033 = 36139 \text{ t/ft} = 3012 \text{ t/in}$$

Meanwhile, the stiffness constant k_{wg} for the pile group is as follows.

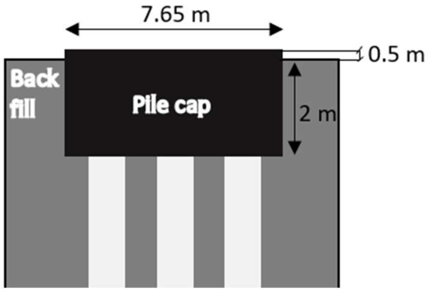


Figure 5.25 Front View of Pile Foundation Group Cut

It can be seen from the figure that as much as 2 m (6.56 ft) of the pile cap is embedded ($h_{embedded}$), while the remaining 0.5 m (1.64 ft) is above the back fill.

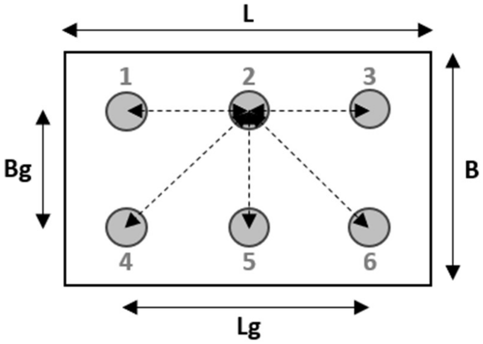


Figure 5.26 Pile Numbering and Pile Reference

According to Figure 3.19, the value of α_A as a function of pile length and spacing is determined. Using Pile 2 as the reference and the v value of 0.5, the α_A value is determined as follows.

1. For the reference pile (Pile 2), the α_A value is 1.
2. For the adjacent piles (Piles 1, 3, 5), the α_A value is determined as follows.

   Spacing of piles (s) = 2.55 m = 8.37 ft

   $s/2r_0 = 8.37/2(1.39) = 3$

   $L/2r_0 = 49.21/2(1.39) = 17.65$

   $L/2r_0 = 10$ $\alpha_A = 0.48$

   $L/2r_0 = 25$ $\alpha_A = 0.56$

   The values are then interpolated to determine the α_A value with $L/2r_0$ equal to 17.65. The interpolation result is as follows.

   $L/2r_0 = 17.65$ $\alpha_A = 0.52$

3. For the diagonal piles (Piles 4, 6), the α_A value is determined as follows.

Spacing of piles (s) = $\sqrt{8.37^2 + 8.37^2}$ = 11.83 ft

$s/2r_0$ = 11.83/2(1.39) = 4.24

$L/2r_0$ = 49.21/2(1.39) = 17.65

$L/2r_0 = 10$ $\alpha_A = 0.38$

$L/2r_0 = 25$ $\alpha_A = 0.48$

The values are then interpolated to determine the α_A value with $L/2r_0$ equal to 17.65. The interpolation result is as follows.

$L/2r_0 = 17.65$ $\alpha_A = 0.43$

After obtaining the values of α_A, the total value of α_A is determined as follows.

$\Sigma\alpha_A$ = 1 + 3(0.52) + 2(0.43) = 3.42

The combined spring stiffness constant k_{wg} of piles in a vertical direction for the homogeneous soil profile according to Equation 3.170 is calculated as follows.

$$k_w^g = \frac{n \cdot k_w^1}{\Sigma\alpha_A} = \frac{6 \cdot 3468}{3.42} = 6076.37\ \mathrm{t/in}$$

The combined spring stiffness constant k_{wg} of piles in a vertical direction for the parabolic soil profile is as follows.

$$k_w^g = \frac{n \cdot k_w^1}{\Sigma\alpha_A} = \frac{6 \cdot 3012}{3.42} = 5276.84\ \mathrm{t/in}$$

The spring stiffness due to side friction of pile cap k_{wf} is also taken into consideration. The analysis according to Equation 3.171 is as follows.

$$\overline{S_1} = 2.7$$

$$k_w^f = Gs \cdot h_{embedded} \cdot \overline{S_1} = 400 \cdot 6.56 \cdot 2.7 = 7086.96\ \mathrm{t/ft} = 590.58\ \mathrm{t/in}$$

Therefore, the total spring stiffness constant k_w for a vertically vibrating pile of the homogeneous soil profile is determined according to Equation 3.169 as follows.

$$k_w = k_w^g + k_w^f = 6076.37 + 590.58 = 6666.95\ \mathrm{t/in}$$

Meanwhile, the total spring stiffness constant k_w for a vertically vibrating pile of the parabolic soil profile is determined as follows.

$$k_w = k_w^g + k_w^f = 5276.84 + 590.58 = 5867.42\ \mathrm{t/in}$$

5.7.2 Lateral Vibration

The initial data needed to analyze lateral vibration is as follows.

EI = 1.2×10^{10} lb-in^2

E_{pile} = 2.5×10^5 t/ft^2 = 1.7×10^3 t/in^2 = 3.5×10^6 lb/in^2

I = EI/E = $(1.2\times10^{10})/(3.5\times10^6) = 3.5\times10^3$ in^4

Gs = 400 t/ft^2

Soil shear modulus around pile cap is reduced to 60% of original:

60%Gs= 240 t/ft^2

Soil shear modulus around pile (itself) is reduced to 75% of original:

75%Gs= 300 t/ft^2

G_{soil} = constant with depth

In the analysis of lateral vibration, the parameters of horizontal response for piles with $L/r_0 > 25$ for homogenous soil profile are obtained according to Table 3.17. The horizontal (sliding) stiffness (f_{x1}) parameters of the homogeneous soil profile are determined as follows.

Ep/Gs = 250000/300 = 833

v (Poisson's ratio) = 0.4

Ep/Gs = 1000 f_{x1} = 0.0261

Ep/Gs = 500 f_{x1} = 0.0436

The values are then interpolated to determine the f_{x1} value with Ep/Gs equal to 833. The interpolation result is as follows.

Ep/Gs = 833 f_{x1} = 0.0378

Meanwhile, the parameters of horizontal response for piles with $L/r_0 > 30$ for parabolic soil profile are obtained according to Table 3.17. The horizontal (sliding) stiffness (f_{x1}) parameters of the parabolic soil profile are determined as follows.

Ep/Gs = 250000/300 = 833

v (Poisson's ratio) = 0.4

Ep/Gs = 1000 f_{x1} = 0.0094

Ep/Gs = 500 f_{x1} = 0.0149

The values are then interpolated to determine the f_{x1} value with Ep/Gs equal to 833. The interpolation result is as follows.

Ep/Gs = 833 f_{x1} = 0.0131

The spring stiffness constant k_x of one pile in a lateral direction for the homogeneous soil profile according to Equation 3.172 is calculated as follows.

$$k_x^1 = \frac{E_p \cdot I_p}{r_0{}^3}(f_x^1) = \frac{1.7 \cdot 10^3 \cdot 3.5 \cdot 10^3}{16.73^3}(0.0378) = 48.37 \text{ t/in}$$

The spring stiffness constant k_x of one pile in a lateral direction for the parabolic soil profile is as follows.

$$k_x^1 = \frac{E_p \cdot I_p}{r_0{}^3}(f_x^1) = \frac{1.7 \cdot 10^3 \cdot 3.5 \cdot 10^3}{16.73^3}(0.0131) = 16.74 \text{ t/in}$$

Meanwhile, the stiffness constant k_{xg} for the pile group is as follows. To analyze horizontal translation, the values of α_L are needed, which can be obtained from Figure 3.20 using the dotted lines for flexible pile. Using Pile 2 as the reference, the α_L value is determined as follows.

1. For the reference pile (Pile 2), the α_L value is 1.
2. For the adjacent piles (Piles 1, 3, 5), the α_L value is determined as follows.

   β = 0°

   Pile stiffness K_R for flexible pile:

   $$K_R = \frac{EI_{pile}}{2(G(1+v))_{soil} \cdot L^4} \cdot 10^{-5} = \frac{4.17 \cdot 10^4}{2(300(1+0.4))_{soil} \cdot 49.21^4} \cdot 10^{-5} = 8.46 \cdot 10^{-11}$$

   Spacing of piles (s) = 2.55 m = 8.37 ft

   $s/2r_0$ = 8.37/2(1.39) = 3

   $s/2r_0$ = 2 α_L = 0.6

   $s/2r_0$ = 5 α_L = 0.3

   The values are then interpolated to determine the α_L value with $s/2r_0$ equal to 3. The interpolation result is as follows.

   $s/2r_0$ = 3 α_L = 0.50
3. For the diagonal piles (Piles 4, 6), the α_L value is determined as follows.

   β = 0°

   Spacing of piles (s) = $\sqrt{8.37^2 + 8.37^2}$ = 11.83 ft

   $s/2r_0$ = 11.83/2(1.39) = 4.24

   $s/2r_0$ = 2 α_L = 0.6

   $s/2r_0$ = 5 α_L = 0.3

The values are then interpolated to determine the α_L value with $s/2r_0$ equal to 4.24. The interpolation result is as follows.

$s/2r_0 = 4.24$ $\alpha_L = 0.38$

After obtaining the values of α_L, the total value of α_L is determined as follows.

$\Sigma\alpha_L = 1 + 3(0.50) + 2(0.38) = 3.25$

The combined spring stiffness constant k_{xg} of piles in a horizontal direction for the homogeneous soil profile according to Equation 3.174 is calculated as follows.

$$k_x^g = \frac{n \cdot k_x^1}{\Sigma\alpha_L} = \frac{6 \cdot 48.37}{3.25} = 89.26 \text{ t/in}$$

The combined spring stiffness constant k_{xg} of piles in a horizontal direction for the parabolic soil profile is as follows.

$$k_x^g = \frac{n \cdot k_x^1}{\Sigma\alpha_L} = \frac{6 \cdot 16.74}{3.25} = 30.88 \text{ t/in}$$

The spring stiffness due to side friction of pile cap k_{xf} is also taken into consideration. The analysis according to Equation 3.175 is as follows.

$\overline{S_{x1}} = 4.1$ according to Table 3.18.

$$k_x^f = Gs \cdot h_{embedded} \cdot \overline{S_{x1}} = 240 \cdot 6.56 \cdot 4.1 = 6457.01 \text{ t/ft} = 538.08 \text{ t/in}$$

Therefore, the total spring stiffness constant k_x for a laterally vibrating pile of the homogeneous soil profile is determined according to Equation 3.173 as follows.

$$k_x = k_x^g + k_x^f = 89.26 + 538.08 = 627.35 \text{ t/in}$$

Meanwhile, the total spring stiffness constant k_x for a laterally vibrating pile of the parabolic soil profile is determined as follows.

$$k_x = k_x^g + k_x^f = 30.88 + 538.08 = 568.97 \text{ t/in}$$

5.7.3 Rocking Vibration

In the analysis of rocking vibration, the parameters of horizontal response for piles with $L/r_0 > 25$ for homogenous soil profile are obtained according to Table 3.17. The rocking stiffness ($f_{\phi 1}$) parameters of the homogenous soil profile are determined as follows.

Ep/Gs = 250000/300 = 833

v (Poisson's ratio) = 0.4

Ep/Gs = 1000 $f_{\phi1}$ = 0.3860
Ep/Gs = 500 $f_{\phi1}$ = 0.4547

The values are then interpolated to determine the $f_{\phi1}$ value with Ep/Gs equal to 833. The interpolation result is as follows.

Ep/Gs = 833 $f_{\phi1}$ = 0.4318

Meanwhile, the parameters of horizontal response for piles with $L/r_0 > 30$ for parabolic soil profile are obtained according to Table 3.17. The rocking stiffness ($f_{\phi1}$) parameters of the parabolic soil profile are determined as follows.

Ep/Gs = 250000/300 = 833
v (Poisson's ratio) = 0.4
Ep/Gs = 1000 $f_{\phi1}$ = 0.3094
Ep/Gs = 500 $f_{\phi1}$ = 0.3596

The values are then interpolated to determine the $f_{\phi1}$ value with Ep/Gs equal to 833. The interpolation result is as follows.

Ep/Gs = 833 $f_{\phi1}$ = 0.3429

The spring stiffness constant k_ϕ of one pile in a lateral direction for the homogeneous soil profile according to Equation 3.176 is calculated as follows.

$$k_\phi^1 = \frac{E_p \cdot I_p}{r_0}\left(f_\phi^1\right) = \frac{1.7 \cdot 10^3 \cdot 3.5 \cdot 10^3}{16.73}(0.4318) = 154838 \text{ in-t/rad}$$

The spring stiffness constant k_ϕ of one pile in a lateral direction for the parabolic soil profile is as follows.

$$k_\phi^1 = \frac{E_p \cdot I_p}{r_0}\left(f_\phi^1\right) = \frac{1.7 \cdot 10^3 \cdot 3.5 \cdot 10^3}{16.73}(0.3429) = 122948 \text{ in-t/rad}$$

Meanwhile, the cross-spring stiffness of single pile ($f_{x\phi1}$) parameters of the homogeneous soil profile are determined as follows.

Ep/Gs = 250000/300 = 833
v (Poisson's ratio) = 0.4
Ep/Gs = 1000 $f_{x\phi1}$ = -0.0714
Ep/Gs = 500 $f_{x\phi1}$ = -0.0991

The values are then interpolated to determine the $f_{x\phi1}$ value with Ep/Gs equal to 833. The interpolation result is as follows.

Ep/Gs = 833 $f_{x\phi1}$ = -0.0806

On the other hand, the cross-spring stiffness of single pile ($f_{x\phi 1}$) parameters of the parabolic soil profile are determined as follows.

Ep/Gs = 250000/300 = 833
v (Poisson's ratio) = 0.4
Ep/Gs = 1000 $f_{x\phi 1}$ = -0.0426
Ep/Gs = 500 $f_{x\phi 1}$ = -0.0577

The values are then interpolated to determine the $f_{x\phi 1}$ value with Ep/Gs equal to 833. The interpolation result is as follows.

Ep/Gs = 833 $f_{x\phi 1}$ = -0.0476

The cross-coupled rocking stiffness constant $k_{x\phi 1}$ of one pile of the homogeneous soil profile according to Equation 3.177 is calculated as follows.

$$k_{x\phi}^{1} = \frac{E_p \cdot I_p}{r_0^2}\left(f_{x\phi}^{1}\right) = \frac{1.7\cdot 10^3\cdot 3.5\cdot 10^3}{16.73^2}(-0.0806) = -1.7\cdot 10^3\ \mathrm{t/in}$$

The cross-coupled rocking stiffness constant $k_{x\phi 1}$ of one pile of the parabolic soil profile is as follows.

$$k_{x\phi}^{1} = \frac{E_p \cdot I_p}{r_0^2}\left(f_{x\phi}^{1}\right) = \frac{1.7\cdot 10^3\cdot 3.5\cdot 10^3}{16.73^2}(-0.0476) = -1.0\cdot 10^3\ \mathrm{t/in}$$

Furthermore, to calculate rocking stiffness due to pile group, the values of the coordinate of pile or the critical depth below ground level (x_r) and the height of center of gravity of pile cap above its base (z_c) need to be determined.

x_r = 0 ft = 0 in for Piles 2 and 5
x_r = 8.37 ft = 100.40 in for Piles 1, 4, 3, and 6
Σx_r = 0 + 100.40 = 100.40 in
z_c = t/2 = 8.20/2 = 4.10 ft = 49.22 in

The calculation of rocking stiffness due to pile group $k_{\phi g}$ for the homogeneous soil profile according to Equation 3.178 is calculated as follows.

$$k_{\phi}^{g} = \sum_1^n\left(k_{\phi}^{1} + k_{w}^{1}\cdot x_r{}^2 + k_{x}^{1}\cdot z_c{}^2 - 2\cdot z_c\cdot k_{x\phi}^{1}\right)$$

$$k_{\phi}^{g} = 6\left(154838 + 3468\cdot 100.4^2 + 48.37\cdot 49.22^2 - 2\cdot 49.22\cdot(-1.7\cdot 10^3)\right)$$

$$k_{\phi}^{g} = 2713035.24\ \mathrm{t/in}$$

The calculation of rocking stiffness due to pile group $k_{\phi g}$ for the parabolic soil profile is as follows.

$$k_\phi^g = \sum_1^n \left(k_\phi^1 + k_w^1 \cdot {x_r}^2 + k_x^1 \cdot {z_c}^2 - 2 \cdot z_c \cdot k_{x\phi}^1\right)$$

$$k_\phi^g = 6\left(122948 + 3012 \cdot 100.4^2 + 16.74 \cdot 49.22^2 - 2 \cdot 49.22 \cdot (-1.0 \cdot 10^3)\right)$$

$$k_\phi^g = 1644264.95 \text{ t/in}$$

The rocking stiffness due to side friction of pile cap $k_{\phi f}$ is also taken into consideration. The analysis according to Equations 3.179 and 3.180 is as follows.

$\overline{S_{\phi 1}} = 2.5$ and $\overline{S_{x1}} = 4.1$ according to Table 3.18.

$$\delta = \frac{h}{r_0} = \frac{6.56}{11.56} = 0.57$$

$$k_\phi^f = G_s \cdot r_0^2 \cdot h \cdot \overline{S_{\phi 1}} + G_s \cdot r_0^2 \cdot h \left[\left(\frac{\delta^2}{3}\right) + \left(\frac{z_c}{r_0}\right)^2 - \delta\left(\frac{z_c}{r_0}\right)\right] \overline{S_{x1}}$$

$$k_\phi^f = 240 \cdot 11.56^2 \cdot 6.56 \times \left\{2.5 + \left[\left(\frac{0.57^2}{3}\right) + \left(\frac{4.10}{11.56}\right)^2 - 0.57\left(\frac{4.10}{11.56}\right)\right] 4.1\right\}$$

$$k_\phi^f = 46156 \text{ t/in}$$

Therefore, the total spring stiffness constant k_ϕ for a rocking pile of the homogeneous soil profile is determined according to Equation 3.181 as follows.

$$k_\phi = k_\phi^g + k_\phi^f = 2713035.24 + 46156 = 2759191 \text{ t/in}$$

Meanwhile, the total spring stiffness constant k_ϕ for a rocking pile of the parabolic soil profile is determined as follows.

$$k_\phi = k_\phi^g + k_\phi^f = 1644264.95 + 46156 = 1690421 \text{ t/in}$$

Finally, the spring stiffness results of all the vibration types for the homogeneous soil profile are summarized in the following table.

Table 5.87 Spring Stiffness Result for Flexible Foundation with Homogeneous Soil Profile

Vibration Type	Spring Stiffness			
Vertical	Single pile	k_{w1}	3467.90	t/in
			1339372.89	kN/m
	Pile group	k_{wg}	6076.37	t/in
			2346816.90	kN/m
	Pile cap side friction	k_{wf}	590.58	t/in
			228094.09	kN/m
	Total stiffness	Total k_w	6666.95	t/in
			2574910.99	kN/m
Lateral	Single pile	k_{x1}	48.37	t/in
			18682.26	kN/m

	Pile group	k_{xg}	89.26	t/in
			34475.63	kN/m
	Pile cap side friction	k_{xf}	538.08	t/in
			207819.06	kN/m
	Total stiffness	Total k_x	627.35	t/in
			242294.68	kN/m
Rocking	Single pile	$k_{\phi 1}$	154838.40	t-in/rad
			38581.70	kN-m/rad
	Cross-coupled	$k_{x\phi 1}$	-1728.05	t
			-16952.14	kN
	Pile group	$k_{\phi g}$	2713035.24	t-in/rad
			676017.84	kN-m/rad
	Pile cap side friction	$k_{\phi f}$	46156.06	t-in/rad
			11500.89	kN-m/rad
	Total stiffness	Total k_ϕ	2759191.29	t-in/rad
			687518.73	kN-m/rad

The values of spring stiffness for the homogeneous soil profile that are inserted into the ETABS model analysis are the total stiffness values, which are as follows.

Translation Z = 2574910.99 kN/m

Translation X, Y = 242294.68 kN/m

Rotation X, Y, Z = 687518.73 kN-m/rad

Meanwhile, as a comparison, the spring stiffness results of all the vibration types for the parabolic soil profile are summarized in the following table.

Table 5.88 Spring Stiffness Result for Flexible Foundation with Parabolic Soil Profile

Vibration Type	Spring Stiffness			
Vertical	Single pile	kw1	3011.59	t/in
			1163139.61	kN/m
	Pile group	kwg	5276.84	t/in
			2038025.21	kN/m
	Pile cap side friction	kwf	590.58	t/in
			228094.09	kN/m
	Total stiffness	Total kw	5867.42	t/in
			2266119.29	kN/m
Lateral	Single pile	kx1	16.74	t/in
			6463.77	kN/m
	Pile group	kxg	30.88	t/in
			11928.02	kN/m
	Pile cap side friction	kxf	538.08	t/in
			207819.06	kN/m

	Total stiffness	Total kx	568.97	t/in
			219747.08	kN/m
Rocking	Single pile	kϕ1	122947.95	t-in/rad
			30635.43	kN-m/rad
	Cross-coupled	kxϕ1	-1020.83	t
			-10014.30	kN
	Pile group	kϕg	1644264.95	t-in/rad
			409708.08	kN-m/rad
	Pile cap side friction	kϕf	46156.06	t-in/rad
			11500.89	kN-m/rad
	Total stiffness	Total kϕ	1690421.01	t-in/rad
			421208.97	kN-m/rad

In this study, the spring stiffness values that are inserted into the ETABS model analysis are the ones with homogeneous soil profile, therefore the purpose of the values with parabolic soil profile is for a comparison. The results show that the homogeneous soil profile produces relatively higher values of spring stiffness compared to the parabolic soil profile.

5.9 Fundamental Period Analysis

The fundamental period of the building is determined after analyzing the approach fundamental period (Ta) and the upper bound of the calculated period (CuTa) values. The previously determined values of Ta and CuTa are as follows.

Ta = 1.86 s

CuTa = 2.60 s

Meanwhile, the obtained fundamental period value from the ETABS model with fixed support is 2.445 s. The fundamental period of fixed support is then compared with the fundamental period of flexible support. Additionally, the building is also analyzed using pin/hinge support as a comparison, as most building designs are usually initially modeled using a pin/hinge support. The fundamental period comparison can be seen in the following table.

Table 5.89 Fundamental Period Comparison According to Support (Foundation) Types

Parameter	Structural Systems			
	Fixed Base	Flexible Base		Pin/Hinge Base
		Homogeneous Soil Profile	Parabolic Soil Profile	
Fundamental Period T (s)	2.445	2.602	2.646	2.789

To confirm the values of the fundamental period, the flexible support model with 0 stiffness values must have approximately the same fundamental period as pin support. Meanwhile, the flexible support model with infinity stiffness values must have approximately the same fundamental period as fixed support model. To test this, new models with 0 and infinity stiffness values are analyzed to determine the fundamental period values. The results are as follows.

Tc for 0 stiffness = 2.789 s
Tc for infinity stiffness = 2.445 s

The fundamental period of a flexible support with 0 stiffness is 2.789 s, which is the same value as the fundamental period of a pin support. Moreover, the fundamental period of a flexible support with infinity stiffness is 2.445 s, which is the same value as the fundamental period of a fixed support.

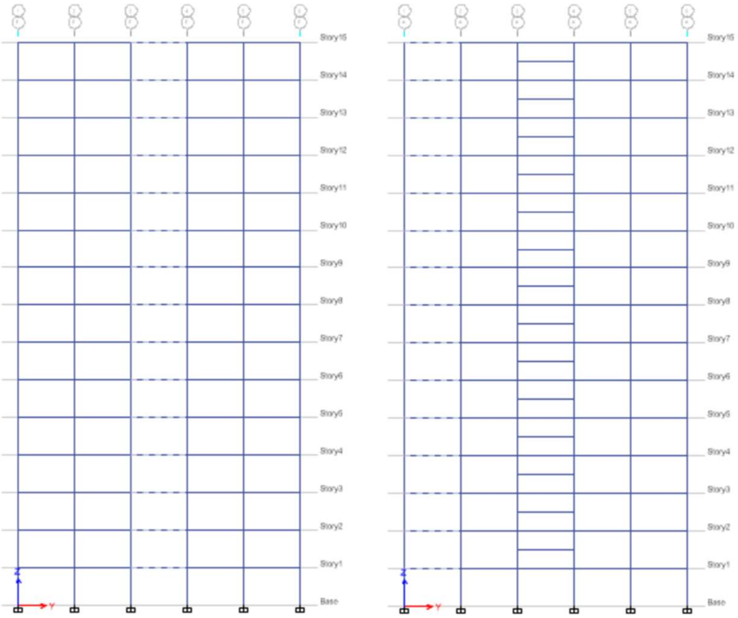
As the fundamental period values show that the requirements are fulfilled, it can be concluded that the analysis of the fundamental period is completed.

5.10 Internal Forces

The internal forces considered in this analysis include the shear force and flexural moment of beams and columns, drift ratio, joint rotation, and horizontal joint displacement.

5.9.1 Shear Force

Shear force analysis is applied to both beams and columns. The earthquake direction considered in this analysis is the Y direction, as it is considered the more vulnerable direction for the building model. For beams, the samples taken are in axes F and A.



(a) (b)

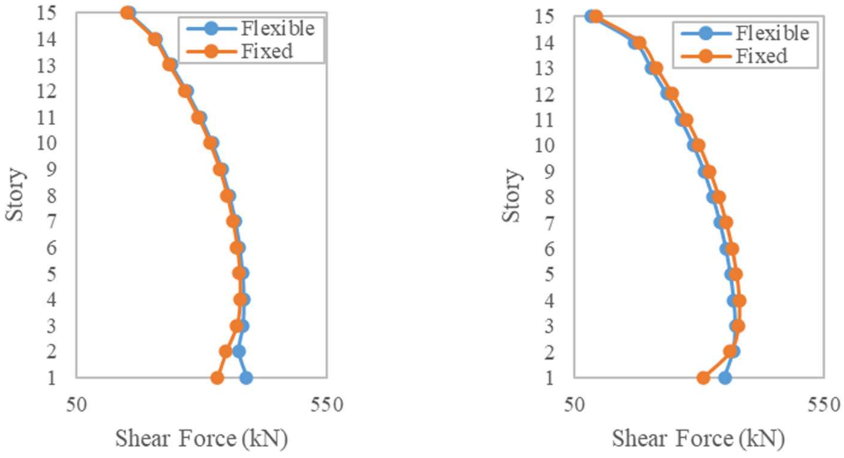
Figure 5.27 Sample Location of Beams in (a) Axis F and (b) Axis A

The values of shear forces in each beam of each story are obtained from the ETABS model analysis. The shear force results for beams in axes F and A with fixed and flexible foundations are as follows.

Table 5.90 Shear Force Result of Beams in Axes F and A

Story	Beam Shear Force (kN)			
	Axis F		Axis A	
	Fixed	Flexible	Fixed	Flexible
15	151.19	155.05	93.16	83.70
14	204.55	209.08	180.62	169.98
13	234.65	239.05	213.41	202.90
12	265.13	269.59	245.04	234.38
11	292.26	296.74	273.93	263.16
10	315.41	319.94	299.04	288.11
9	334.48	339.08	320.42	309.30
8	349.70	354.39	338.31	326.98
7	361.35	366.19	353.03	341.52
6	369.63	374.79	364.84	353.25
5	374.53	380.38	373.74	362.44
4	375.28	382.92	378.95	368.95
3	369.09	381.23	377.71	372.06
2	348.00	373.09	361.51	367.31
1	330.81	388.61	307.71	349.90

Meanwhile, the vertical distribution diagrams generated from the shear force data of beams in axes F and A are as follows.



(a) (b)

Figure 5.28 Shear Force Vertical Distribution Diagram of Beams in (a) Axis F and (b) Axis A

As a result, in axis F, the beam shear force vertical distribution diagram of the flexible foundation shows a slightly higher value than the fixed, especially in the lower stories. On the other hand, in axis A, the beam shear force vertical distribution diagram of the fixed foundation shows a slightly higher value than the flexible, except for stories 1 and 2. The difference in results between axis F and A may be caused by the axis location, as axis F is located around the center of the building, while axis A is located in the outermost part of the building.

Meanwhile, for columns, there are 6 samples considered: 3 in axis F and another 3 in axis A. The position of the columns can be seen in the following figure.

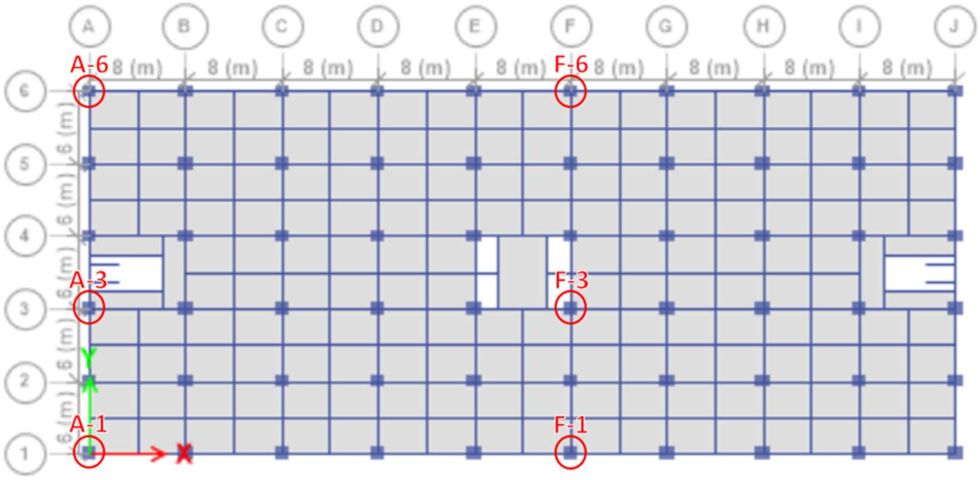


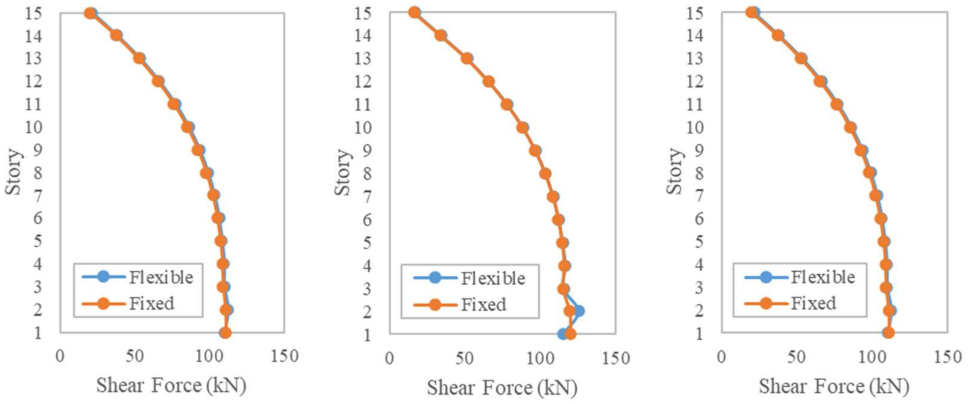
Figure 5.29 Sample Position of Columns

As can be seen in the figure, columns on axis A are located in the outermost corner of the building, while the ones on axis F are located around the middle of the building. The shear force results for columns in axis F with fixed and flexible foundations are as follows.

Table 5.91 Shear Force Result of Columns in Axis F

Story	Column Shear Force (kN)					
	Axis F-1		Axis F-3		Axis F-6	
	Fixed	Flexible	Fixed	Flexible	Fixed	Flexible
15	20.10	21.61	16.06	16.77	20.06	21.58
14	37.38	38.30	33.89	34.33	37.37	38.29
13	52.60	53.65	51.08	51.58	52.58	53.63
12	65.47	66.50	65.63	66.12	65.45	66.48
11	76.28	77.33	78.01	78.50	76.27	77.31
10	85.20	86.25	88.29	88.77	85.19	86.24
9	92.40	93.46	96.65	97.13	92.39	93.45
8	98.06	99.13	103.27	103.76	98.05	99.12
7	102.34	103.42	108.34	108.84	102.34	103.42
6	105.43	106.52	112.03	112.58	105.43	106.52
5	107.50	108.58	114.45	115.04	107.50	108.58
4	108.83	109.95	115.94	116.92	108.84	109.95
3	109.25	110.04	115.13	115.74	109.26	110.01
2	110.81	112.52	119.55	125.65	110.93	112.66
1	111.21	110.24	120.21	115.42	111.33	110.30

The vertical distribution diagrams generated from the shear force data of columns in axis F are as follows.



(a) (b) (c)

Figure 5.30 Shear Force Vertical Distribution Diagram of Columns (a) F-1, (b) F-3, and (c) F-6

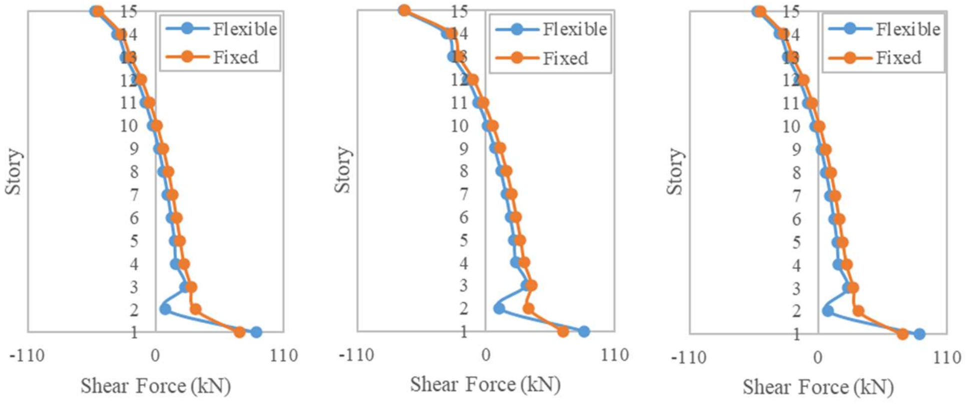
As a result, it can be seen from the vertical distribution diagram that the shear force results of columns F-1 and F-6 are rather similar because they are both located on the outermost part of the building. In addition, in axis F, the values of the shear force of the building designed with fixed and flexible foundations are not noticeably distinctive.

Meanwhile, the shear force results for columns in axis A with fixed and flexible foundations are as follows.

Table 5.92 Shear Force Result of Columns in Axis A

Story	Column Shear Force (kN)					
	Axis A-1		Axis A-3		Axis A-6	
	Fixed	Flexible	Fixed	Flexible	Fixed	Flexible
15	-48.90	-51.87	-69.15	-70.61	-48.87	-51.85
14	-29.79	-33.33	-29.41	-33.46	-29.79	-33.33
13	-21.74	-25.83	-23.14	-27.78	-21.75	-25.85
12	-12.11	-16.12	-10.86	-15.42	-12.12	-16.13
11	-4.79	-8.86	-2.05	-6.69	-4.80	-8.87
10	1.48	-2.63	5.71	1.00	1.47	-2.64
9	6.69	2.51	12.16	7.36	6.68	2.50
8	11.05	6.78	17.57	12.65	11.05	6.78
7	14.75	10.33	22.11	17.04	14.74	10.33
6	17.96	13.26	25.97	20.62	17.96	13.26
5	21.09	16.08	29.55	23.99	21.09	16.08
4	24.35	17.61	32.63	25.55	24.36	17.61
3	30.31	25.17	38.73	34.18	30.30	25.13
2	34.16	8.47	36.01	11.10	34.25	8.58
1	71.81	86.25	65.55	83.45	71.92	86.31

The vertical distribution diagrams generated from the shear force data of columns in axis A are as follows.



(a) (b) (c)

Figure 5.31 Shear Force Vertical Distribution Diagram of Columns (a) A-1, (b) A-3, and (c) A-6

As a result, similarly to the columns in axis F, it can be seen from the vertical distribution diagram that the shear force results of columns A-1 and A-6 are rather similar because they are both located on the outermost part of the building. In addition, in axis A, the values of the shear force of the building designed with a fixed foundation are noticeably slightly higher than the one designed with a flexible foundation.

To confirm the result of the flexible foundation having a bigger shear force value, the maximum beam shear force values of both X and Y directions are analyzed and compared. Moreover, the base shear force in response spectrum analysis (RSA) is also analyzed and compared. The comparison is as follows.

Table 5.93 Shear Force Comparison Between Fixed and Flexible Foundation

Parameters	Structural Systems		Remark	Difference
	Fixed Base	Flexible Base		
RSA Base Shear Force (kN)				
X Direction	12445.10	12183.30	(–)	2.10%
Y Direction	12591.01	12292.26	(–)	2.37%
Max Beam Shear Force (kN)				
X Direction	440.74	435.13	(–)	1.27%
Y Direction	428.29	436.23	(+)	1.85%
Max Column Shear Force (kN)				

X Direction	425.66	432.78	(+)	1.67%
Y Direction	203.60	215.97	(+)	6.08%
Remark: (+) Increase, (–) Decrease, (=) Equal, (.) Unclear				

From the table, the RSA base shear force values in flexible foundations are smaller compared to fixed foundations. For the maximum beam and column shear force, the flexible foundation values in the Y direction are indeed bigger than the fixed one. However, in the X direction, the values of the fixed foundation are bigger than the flexible one. Finally, for the maximum column shear force, the flexible foundation indeed has bigger values than the fixed one.

5.9.2 Flexural Moment

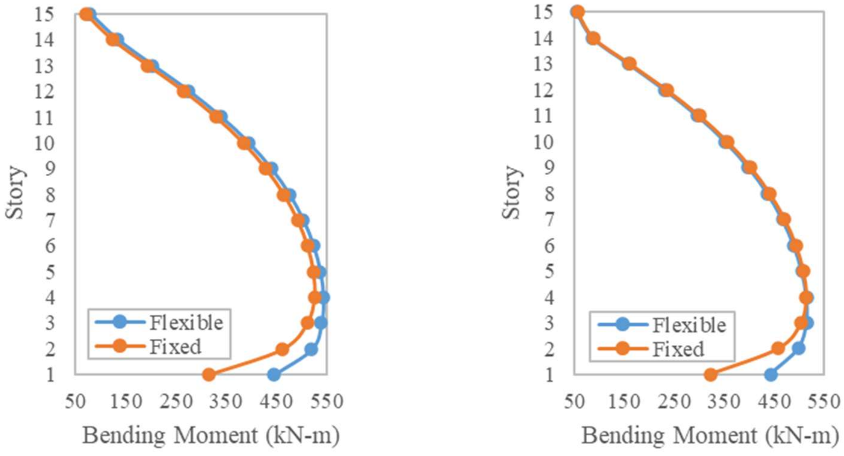
Similar to shear force, the flexural or bending moment analysis is also applied to both beams and columns. The earthquake direction considered in this analysis is the Y direction, as it is considered the more vulnerable direction for the building model. For beams, the samples taken are also in axes F and A.

The values of bending moments in each beam of each story are obtained from the ETABS model analysis. The bending moment results for beams in axes F and A with fixed and flexible foundations are as follows.

Table 5.94 Bending Moment Result of Beams in Axes F and A

Story	Beam Bending Moment (kN-m)			
	Axis F		Axis A	
	Fixed	Flexible	Fixed	Flexible
15	70.30	78.79	56.47	54.09
14	123.51	133.44	89.00	87.18
13	193.89	203.53	161.41	158.65
12	265.47	275.23	234.47	230.78
11	329.14	338.97	299.87	296.07
10	383.51	393.45	355.96	352.02
9	428.35	438.42	402.65	398.58
8	464.17	474.45	440.58	436.37
7	491.63	502.25	470.48	466.24
6	511.26	522.58	492.97	489.00
5	522.98	535.89	508.01	505.20
4	525.04	542.08	513.82	514.66
3	510.95	538.42	503.84	515.09
2	461.74	519.42	458.42	498.48
1	314.40	443.85	322.23	442.18

Meanwhile, the vertical distribution diagrams generated from the bending moment data of beams in axes F and A are as follows.



(a) (b)

Figure 5.32 Bending Moment Vertical Distribution Diagram of Beams in (a) Axis F and (b) Axis A

As a result, in axis F which is located around the center of the building, the beam bending moment vertical distribution diagram of the flexible foundation shows a slightly higher value than the fixed. Meanwhile, in axis A which is located at the outermost part of the building, the beam bending moment vertical distribution diagram of the fixed foundation shows a slightly higher value than the flexible, especially in the lower stories.

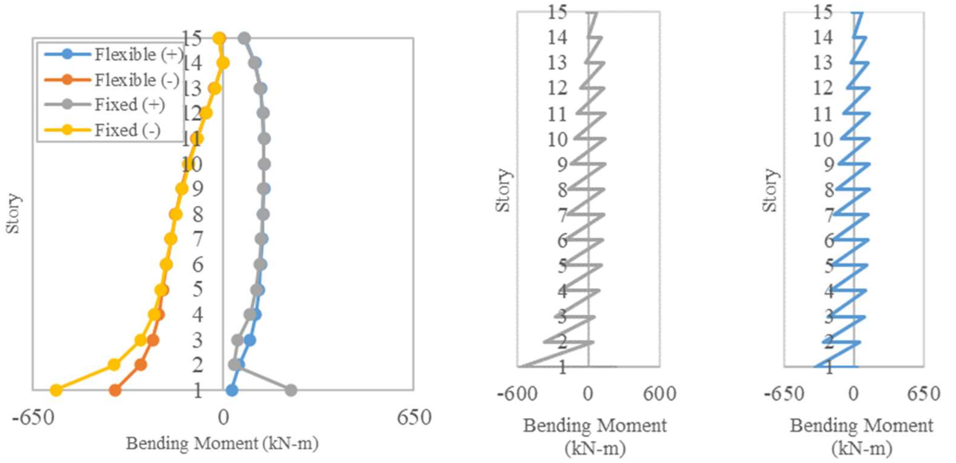
As a result, in axis F, the beam bending moment vertical distribution diagram of the flexible foundation shows a slightly higher value than the fixed, especially in the lower stories. On the other hand, in axis A, the beam bending moment vertical distribution diagram of the fixed foundation shows a slightly higher value than the flexible, except for stories 1 until 4. The difference in results between axis F and A may be caused by the axis location, as axis F is located around the center of the building, while axis A is located in the outermost part of the building.

Meanwhile, for columns, the 6 samples considered are also 3 columns in axis F and another 3 in axis A. The bending moment results for columns in axis F with fixed and flexible foundations are as follows.

Table 5.95 Bending Moment Result of Columns in Axis F

Story	Column Bending Moment (kN-m)											
	Axis F-1				Axis F-3				Axis F-6			
	Fixed		Flexible		Fixed		Flexible		Fixed		Flexible	
	(-)	(+)	(-)	(+)	(-)	(+)	(-)	(+)	(-)	(+)	(-)	(+)
15	-14.96	71.01	-13.18	72.91	-33.64	97.88	-34.00	102.44	-15.05	70.96	-13.25	72.87
14	-1.59	107.68	-0.15	108.46	-29.41	121.63	-33.35	123.45	-1.63	107.69	-0.18	108.46
13	-27.04	127.16	-28.73	128.18	-62.24	145.96	-66.41	148.37	-26.99	127.15	-28.69	128.16
12	-58.48	135.49	-60.14	136.47	-96.45	155.42	-100.60	157.75	-58.44	135.48	-60.10	136.47
11	-88.80	138.80	-90.43	139.83	-129.34	160.40	-133.51	162.82	-88.76	138.79	-90.39	139.83
10	-116.15	139.17	-117.70	140.29	-158.67	162.00	-162.81	164.53	-116.12	139.17	-117.66	140.30
9	-140.01	137.74	-141.38	139.05	-183.99	161.54	-188.04	164.30	-139.98	137.74	-141.35	139.06
8	-160.51	134.91	-161.47	136.63	-205.43	159.38	-209.17	162.60	-160.48	134.91	-161.44	136.64
7	-178.26	130.59	-178.31	133.23	-223.63	155.46	-226.60	159.66	-178.23	130.60	-178.29	133.24
6	-194.60	124.01	-192.61	128.68	-239.84	149.01	-240.99	155.32	-194.57	124.03	-192.59	128.70
5	-212.23	113.04	-205.66	122.23	-256.99	137.85	-253.88	148.83	-212.19	113.08	-205.63	122.26
4	-236.82	92.60	-220.13	111.89	-278.89	116.89	-266.66	138.22	-236.78	92.67	-220.09	111.95
3	-280.76	51.08	-241.01	92.89	-324.63	74.00	-289.30	118.44	-280.65	51.19	-240.80	93.02
2	-370.78	37.77	-280.99	54.44	-393.91	39.17	-321.04	79.49	-370.98	37.61	-281.19	54.69
1	-570.14	233.09	-367.27	29.39	-575.75	226.04	-367.28	22.07	-570.65	233.22	-367.49	29.42

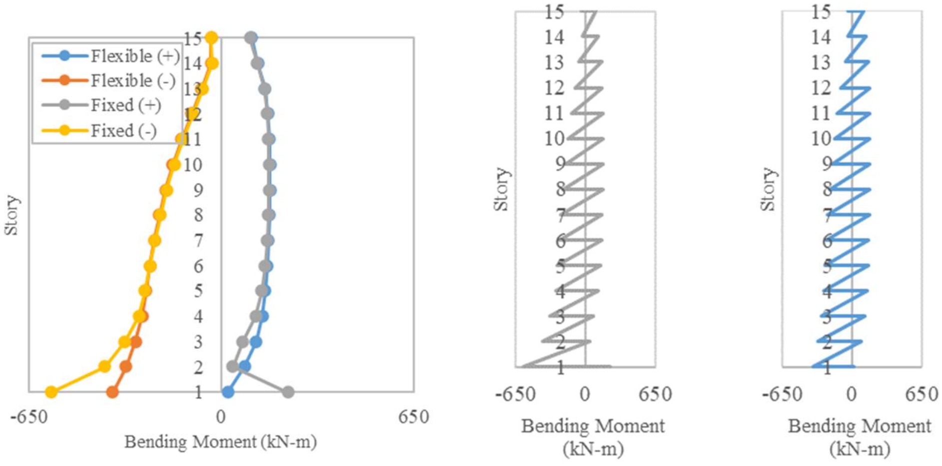
The vertical distribution diagrams generated from the bending moment data of columns in axis F are as follows.



(a)

(b) (c)

Figure 5.33 Bending Moment Vertical Distribution Diagram of Column F-1 (a) Fixed and Flexible Comparison, (b) Fixed, and (c) Flexible

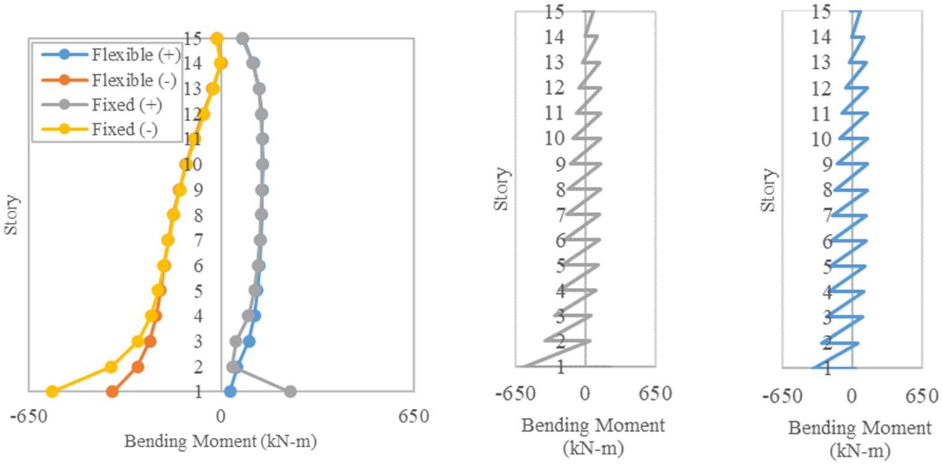


(a)

(b)

(c)

Figure 5.34 Bending Moment Vertical Distribution Diagram of Column F-3 (a) Fixed and Flexible Comparison, (b) Fixed, and (c) Flexible



(a)

(b)

(c)

Figure 5.35 Bending Moment Vertical Distribution Diagram of Column F-6 (a) Fixed and Flexible Comparison, (b) Fixed, and (c) Flexible

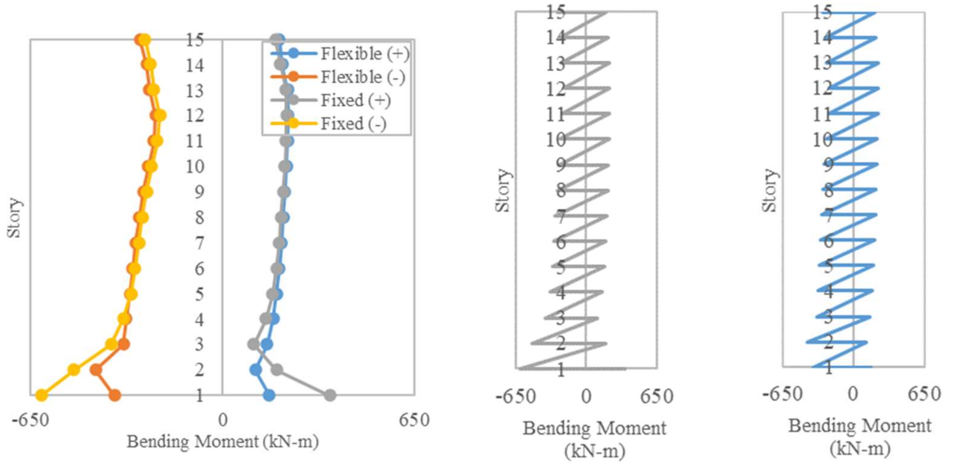
As a result, it can be seen from the vertical distribution diagram that the bending moments of columns F-1 and F-6 are rather similar because they are both located in the outermost corner part of the building. In addition, in axis F, the values of the bending moment of the building designed with fixed and flexible foundations are not distinctive in the upper stories but have a noticeable difference in the lower stories.

Meanwhile, the bending moment results for columns in axis A with fixed and flexible foundations are as follows.

Table 5.96 Bending Moment Result of Columns in Axis A

Story	Column Bending Moment (kN-m)											
	Axis A-1				Axis A-3				Axis A-6			
	Fixed		Flexible		Fixed		Flexible		Fixed		Flexible	
	(-)	(+)	(-)	(+)	(-)	(+)	(-)	(+)	(-)	(+)	(-)	(+)
15	-263.12	178.79	-281.08	191.48	-309.17	243.16	-327.45	253.62	-263.24	178.84	-281.22	191.56
14	-243.81	195.68	-258.09	200.31	-263.92	195.98	-278.50	199.84	-243.88	195.71	-258.18	200.35
13	-232.47	213.62	-247.63	220.02	-252.83	227.68	-268.33	232.98	-232.54	213.65	-247.73	220.06
12	-211.99	215.89	-227.10	221.97	-227.18	226.89	-242.68	231.94	-212.06	215.92	-227.20	222.01
11	-221.45	214.36	-232.57	220.60	-244.82	225.99	-253.96	231.20	-221.47	214.39	-232.62	220.65
10	-240.93	210.11	-252.06	216.47	-263.78	221.63	-272.94	226.97	-240.94	210.14	-252.11	216.52
9	-257.80	204.58	-268.86	211.19	-280.35	216.18	-289.45	221.77	-257.82	204.61	-268.91	211.24
8	-272.17	198.11	-282.90	205.19	-294.55	209.78	-303.34	215.85	-272.18	198.14	-282.94	205.24
7	-284.63	190.63	-294.44	198.65	-306.95	202.40	-314.84	209.43	-284.63	190.66	-294.47	198.70
6	-296.68	181.36	-304.33	191.38	-319.15	193.24	-324.96	202.28	-296.67	181.40	-304.36	191.43
5	-310.94	168.28	-313.08	182.65	-333.32	180.30	-333.39	193.73	-310.91	168.33	-313.08	182.70
4	-335.73	146.51	-327.41	170.37	-360.40	158.59	-350.65	181.54	-335.68	146.57	-327.40	170.45
3	-376.99	105.39	-335.89	150.74	-394.65	117.93	-347.93	162.57	-376.85	105.50	-335.67	150.87
2	-503.38	181.44	-430.53	110.29	-559.62	180.80	-500.03	120.58	-503.53	181.31	-430.71	110.53
1	-615.29	361.76	-367.29	157.51	-638.12	384.67	-366.82	172.21	-615.79	361.92	-367.51	157.60

The vertical distribution diagrams generated from the bending moment data of columns in axis A are as follows.

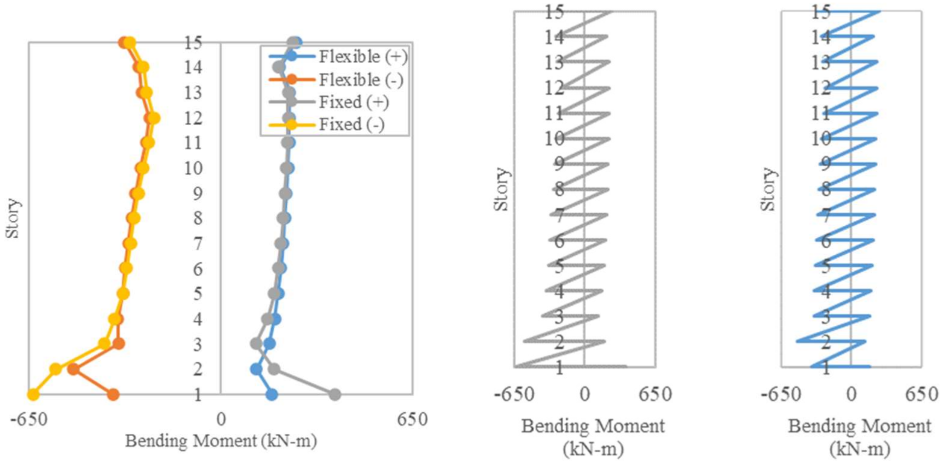


(a)

(b)

(c)

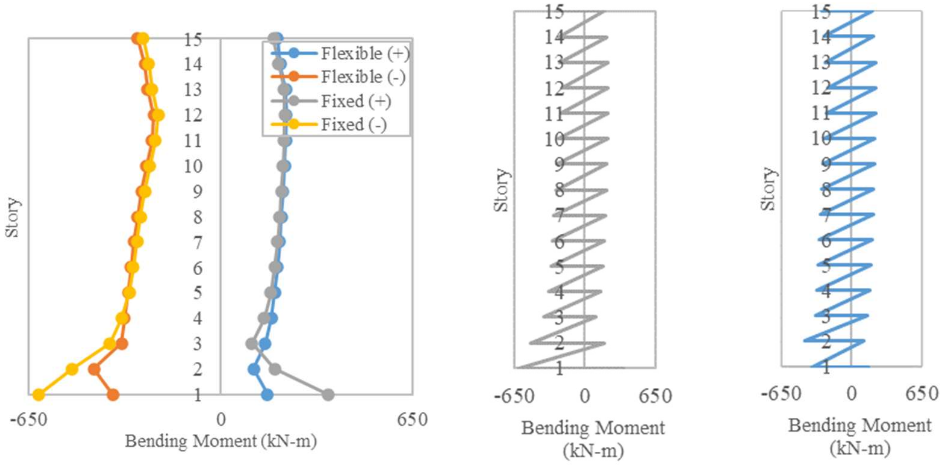
Figure 5.36 Bending Moment Vertical Distribution Diagram of Column A-1 (a) Fixed and Flexible Comparison, (b) Fixed, and (c) Flexible



(a)

(b) (c)

Figure 5.37 Bending Moment Vertical Distribution Diagram of Column A-3 (a) Fixed and Flexible Comparison, (b) Fixed, and (c) Flexible



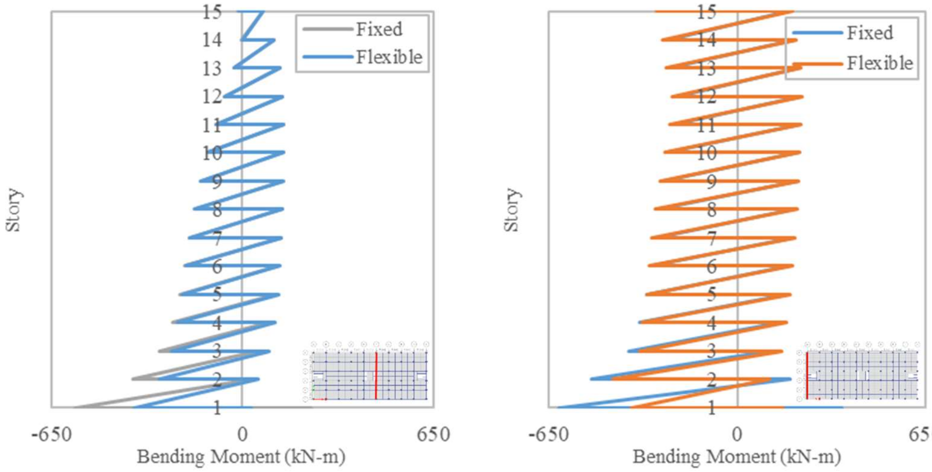
(a)

(b) (c)

Figure 5.38 Bending Moment Vertical Distribution Diagram of Column A-6 (a) Fixed and Flexible Comparison, (b) Fixed, and (c) Flexible

As a result, it can be seen from the vertical distribution diagram that the bending moments of columns A-1 and A-6 are rather similar because they are both located in the outermost corner part of the building. In addition, similarly to axis F, in axis A the values of the bending moment of the building designed with fixed and flexible foundations are not distinctive in the upper stories but have a noticeable difference in the lower stories.

Meanwhile, the bending moment results for columns in both axes with fixed and flexible foundations are as follows.



(a) (b)

Figure 5.39 Bending Moment Vertical Distribution Diagram Comparison Between Columns with Fixed and Flexible Foundation in (a) Axis F and (b) Axis A

As seen from the figure, the bending moments of each story in axis A which is located on the outermost part of the building have a relatively larger value compared to bending moments in axis F which is located around the middle. Additionally, the summary of the bending moment vertical distribution diagram comparison between columns with fixed and flexible foundations in each axis shows that the values are slightly different, especially in the lower stories, where the fixed foundations show larger bending moment values compared to the flexible foundations.

To confirm the result of the flexible foundation having a bigger bending moment value in beams and a smaller bending moment value in columns, the maximum beam and column bending moment values of both X and Y directions are analyzed and compared. The comparison is as follows.

Table 5.97 Bending Moment Comparison Between Fixed and Flexible Foundation

Parameters	Structural Systems		Remark	Difference
	Fixed Base	Flexible Base		
Max Beam Bending Moment (kN-m)				
X Direction	423.64	433.00	(+)	2.21%
Y Direction	532.84	552.70	(+)	3.73%
Max Column Bending Moment (kN-m)				
X Direction	1940.92	1229.76	(–)	36.64%
Y Direction	638.13	505.15	(–)	20.84%
Remark: (+) Increase, (–) Decrease, (=) Equal, (.) Unclear				

Hence, it is proven that the bending moment of beams is bigger in flexible foundations, contrary to having smaller bending moment values in columns.

5.9.3 Drift Ratio

The drift ratio of fixed and flexible foundations in each direction is analyzed to determine whether they have fulfilled the allowable drift requirements. In this analysis, the building is subjected to earthquake load in Y direction (EY). According to SNI 1726:2019, the allowable drift requirement for the type of structure designed and with a risk category of II is $0.020h_{sx}$ or 2% of the height of every story. As the h_{sx} or height of every story is the same, the allowable drift is calculated as follows.

$$\text{Allowable drift} = 0.020 \times \mathrm{h_{sx}} = 0.020 \times 4000 = 80\ \mathrm{mm}$$

Meanwhile, the story drifts and drift ratio calculation examples of fixed foundation drift ratio with earthquake load EY in the X direction are as follows.

$$\Delta_{15} = \frac{(\delta_{15}-\delta_{14})\cdot \mathrm{Cd}}{\mathrm{I_E}} = \frac{(40.015- \quad .064)\cdot 5.5}{1} = 5.23\ \mathrm{mm}$$

$$\text{Drift ratio}_{15} = \frac{\Delta_{15}}{\mathrm{h_{sx}}}\cdot 100\% = \frac{5.23}{4000}\cdot 100\% = 0.131\%$$

The results of the analysis are summarized in the following table.

Table 5.98 Fixed Foundation Drift Ratio Analysis with Earthquake Load EY in X Direction

Story	hsx (mm)	δ (mm)	Δ (mm)	Allowable (mm)	Check	Ratio (%)	Allowable (%)
15	4000	40.02	5.23	80	OK	0.131	2
14	4000	39.06	7.33	80	OK	0.183	2
13	4000	37.73	9.76	80	OK	0.244	2

12	4000	35.96	12.10	80	OK	0.303	2
11	4000	33.76	14.19	80	OK	0.355	2
10	4000	31.18	15.98	80	OK	0.399	2
9	4000	28.27	17.44	80	OK	0.436	2
8	4000	25.10	18.60	80	OK	0.465	2
7	4000	21.72	19.46	80	OK	0.486	2
6	4000	18.18	20.00	80	OK	0.500	2
5	4000	14.55	20.16	80	OK	0.504	2
4	4000	10.88	19.76	80	OK	0.494	2
3	4000	7.29	18.34	80	OK	0.459	2
2	4000	3.95	14.85	80	OK	0.371	2
1	4000	1.25	6.89	80	OK	0.172	2

Meanwhile, the story drifts and drift ratio calculation examples of flexible foundation drift ratio with earthquake load EY in the X direction are as follows.

$$\Delta_{15} = \frac{(\delta_{15} - \delta_{14}) \cdot \mathrm{Cd}}{\mathrm{I_E}} = \frac{(43.68 - 42.70) \cdot 5.5}{1} = 5.39 \text{ mm}$$

$$\text{Drift ratio}_{15} = \frac{\Delta_{15}}{\mathrm{h_{sx}}} \cdot 100\% = \frac{5.39}{4000} \cdot 100\% = 0.135\%$$

The results of the analysis are summarized in the following table.

Table 5.99 Flexible Foundation Drift Ratio Analysis with Earthquake Load EY in X Direction

Story	hsx (mm)	δ (mm)	Δ (mm)	Allowable (mm)	Check	Ratio (%)	Allowable (%)
15	4000	43.68	5.39	80	OK	0.135	2
14	4000	42.70	7.49	80	OK	0.187	2
13	4000	41.34	9.92	80	OK	0.248	2
12	4000	39.54	12.26	80	OK	0.306	2
11	4000	37.31	14.36	80	OK	0.359	2
10	4000	34.70	16.14	80	OK	0.404	2
9	4000	31.76	17.62	80	OK	0.441	2
8	4000	28.56	18.80	80	OK	0.470	2
7	4000	25.14	19.69	80	OK	0.492	2
6	4000	21.56	20.32	80	OK	0.508	2
5	4000	17.87	20.67	80	OK	0.517	2
4	4000	14.11	20.68	80	OK	0.517	2
3	4000	10.35	20.20	80	OK	0.505	2
2	4000	6.67	18.79	80	OK	0.470	2
1	4000	3.26	17.92	80	OK	0.448	2

The comparison diagram between fixed and flexible foundations generated from the drift ratio data with earthquake load EY in the X direction is as follows.

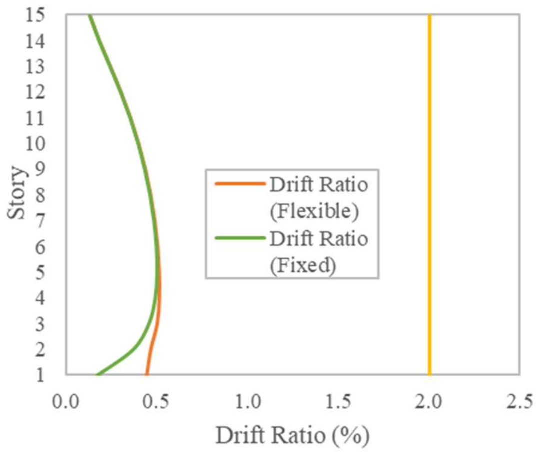


Figure 5.40 Drift Ratio Comparison Diagram Between Fixed and Flexible Foundations with Earthquake Load EY in X Direction

It can be seen from the graph/diagram that the drift ratio with a flexible foundation is slightly higher than the fixed one. Furthermore, both the drift ratios of fixed and flexible foundations have fulfilled the allowable drift ratio requirement.

Meanwhile, the story drifts and drift ratio calculation examples of fixed foundation drift ratio with earthquake load EY in the Y direction are as follows.

$$\Delta_{15} = \frac{(\delta_{15}-\delta_{14})\cdot \mathrm{Cd}}{\mathrm{I_E}} = \frac{(131.39-1\quad .35)\cdot 5.5}{1} = 16.75\ \mathrm{mm}$$

$$\text{Drift ratio}_{15} = \frac{\Delta_{15}}{\mathrm{h_{sx}}} \cdot 100\% = \frac{16.75}{4000} \cdot 100\% = 0.419\%$$

The results of the analysis are summarized in the following table.

Table 5.100 Fixed Foundation Drift Ratio Analysis with Earthquake Load EY in Y Direction

Story	hsx (mm)	δ (mm)	Δ (mm)	Allowable (mm)	Check	Ratio (%)	Allowable (%)
15	4000	131.39	16.75	80	OK	0.419	2
14	4000	128.35	24.13	80	OK	0.603	2
13	4000	123.96	32.28	80	OK	0.807	2
12	4000	118.09	39.88	80	OK	0.997	2
11	4000	110.84	46.48	80	OK	1.162	2
10	4000	102.39	52.01	80	OK	1.300	2
9	4000	92.93	56.49	80	OK	1.412	2
8	4000	82.66	60.00	80	OK	1.500	2
7	4000	71.75	62.58	80	OK	1.565	2
6	4000	60.37	64.31	80	OK	1.608	2
5	4000	48.68	65.06	80	OK	1.626	2

4	4000	36.85	64.46	80	OK	1.612	2
3	4000	25.13	61.21	80	OK	1.530	2
2	4000	14.00	51.63	80	OK	1.291	2
1	4000	4.62	25.38	80	OK	0.635	2

Meanwhile, the story drifts and drift ratio calculation examples of flexible foundation drift ratio with earthquake load EY in the Y direction are as follows.

$$\Delta_{15} = \frac{(\delta_{15}-\delta_{14})\cdot \mathrm{Cd}}{\mathrm{I_E}} = \frac{(143.82-140.49)\cdot 5.5}{1} = 18.29\ \mathrm{mm}$$

$$\text{Drift ratio}_{15} = \frac{\Delta_{15}}{\mathrm{h_{sx}}}\cdot 100\% = \frac{18.29}{4000}\cdot 100\% = 0.457\%$$

The results of the analysis are summarized in the following table.

Table 5.101 Flexible Foundation Drift Ratio Analysis with Earthquake Load EY in Y Direction

Story	hsx (mm)	δ (mm)	Δ (mm)	Allowable (mm)	Check	Ratio (%)	Allowable (%)
15	4000	143.82	18.29	80	OK	0.457	2
14	4000	140.49	25.66	80	OK	0.641	2
13	4000	135.83	33.81	80	OK	0.845	2
12	4000	129.68	41.41	80	OK	1.035	2
11	4000	122.15	48.02	80	OK	1.201	2
10	4000	113.42	53.56	80	OK	1.339	2
9	4000	103.68	58.05	80	OK	1.451	2
8	4000	93.12	61.58	80	OK	1.539	2
7	4000	81.93	64.22	80	OK	1.605	2
6	4000	70.25	66.04	80	OK	1.651	2
5	4000	58.25	67.08	80	OK	1.677	2
4	4000	46.05	67.21	80	OK	1.680	2
3	4000	33.83	65.95	80	OK	1.649	2
2	4000	21.84	61.77	80	OK	1.544	2
1	4000	10.61	58.35	80	OK	1.459	2

The comparison diagram between fixed and flexible foundations generated from the drift ratio data in the X direction is as follows.

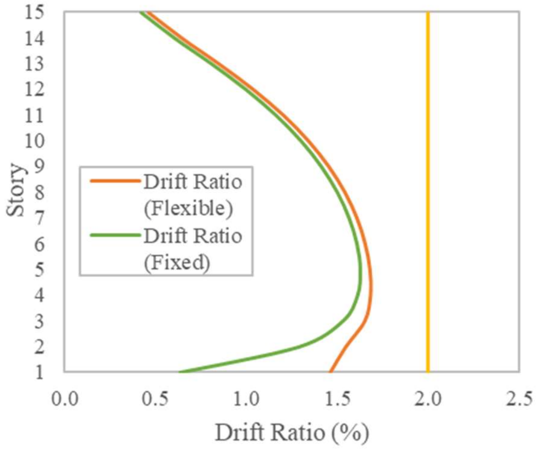


Figure 5.41 Drift Ratio Comparison Diagram Between Fixed and Flexible Foundations with Earthquake Load EY in Y Direction

It can be seen from the graph/diagram that the drift ratio with a flexible foundation is slightly higher than the fixed one. Furthermore, both the drift ratios of fixed and flexible foundations have fulfilled the allowable drift ratio requirement.

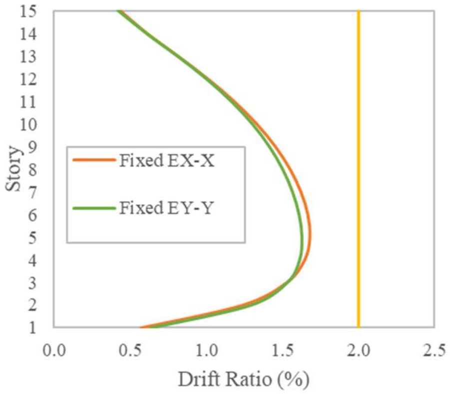
To confirm that flexible foundations create a larger drift ratio value compared to fixed foundations, the maximum drift ratio of both X and Y directions are analyzed and compared. The comparison is as follows.

Table 5.102 Drift Ratio Comparison Between Fixed and Flexible Foundation

Parameters	Structural Systems		Remark	Difference
	Fixed Base	Flexible Base		
Drift Ratio (%)				
X Direction	0.50	0.52	(+)	2.59%
Y Direction	1.63	1.68	(+)	3.31%
Remark: (+) Increase, (–) Decrease, (=) Equal, (.) Unclear				

Hence, it is proven that flexible foundations create a larger drift ratio value compared to fixed foundations.

Additionally, if the drift ratio of the structure is subjected to earthquake load in X direction (EX), it is expected that the drift ratio would be less than when subjected to earthquake load in Y direction (EY). After analyzing the drift ratio with EX load, the results are summarized and compared to drift ratio with EY load in the following figures.



(a) (b)

Figure 5.42 Drift Ratio Comparison Diagram between Earthquake Load EX in X Direction and Earthquake Load EY in Y Direction with (a) Fixed Support and (b) Flexible Support

From the figure above, it is found that with both fixed and flexible support, the drift ratio in X direction when subjected to EX earthquake load is generally slightly greater than the drift ratio in Y direction when subjected to EY earthquake load. This means that the results do not support the logic behind the critical axis of the building.

When taking into consideration the building plan, albeit considered regular and symmetrical, the length of the X axis (72 m) perimeter is far greater than the Y axis (30 m). Logically, this means that the Y axis is more critical when subjected to earthquake load compared to the X axis. Based on the results of drift ratio analysis between EX load in X direction and EY load in Y direction, however, it is found that the X axis shows slightly greater values compared to the Y axis. This may have been caused by several factors, such as:

1. The column orientation was designed to have a greater length in the Y-direction to withstand earthquake load better in the EY-direction, resulting in the X-direction of the building being more vulnerable to a horizontal joint displacement when subjected to earthquake load in the EX-direction—albeit having more columns in the X-axis to withstand the load.
2. The columns or beams might have been designed to have insufficient stiffness/rigidity (especially in the X-direction)—which is directly influenced

by the material properties, cross section, and the length of the structural member.

Aside from the factors mentioned above, it is worth noting that even with the average drift ratio of the X-direction being slightly greater than the Y-direction, some stories generate a higher drift ratio value in some stories. For fixed support, most of the lower stories produce a higher drift ratio in the Y-direction. Meanwhile, for flexible support, the higher drift ratio values are achieved in some of the upper stories.

As the stiffness of the structural member is also listed as a factor, the material properties were included as an influence on the horizontal joint displacement of the stories of the building. The material properties are modulus of elasticity (E) and moment of inertia (I). As the modulus of elasticity value used is constant, the remaining factor should be the moment of inertia. This means that along with cross section and length of the structural member, moment of inertia is also an influential factor in the results of the drift ratio analysis. This suggests an extension of research which may further reveal the factors of the critical axis of the building plan.

5.9.4 Joint Rotation

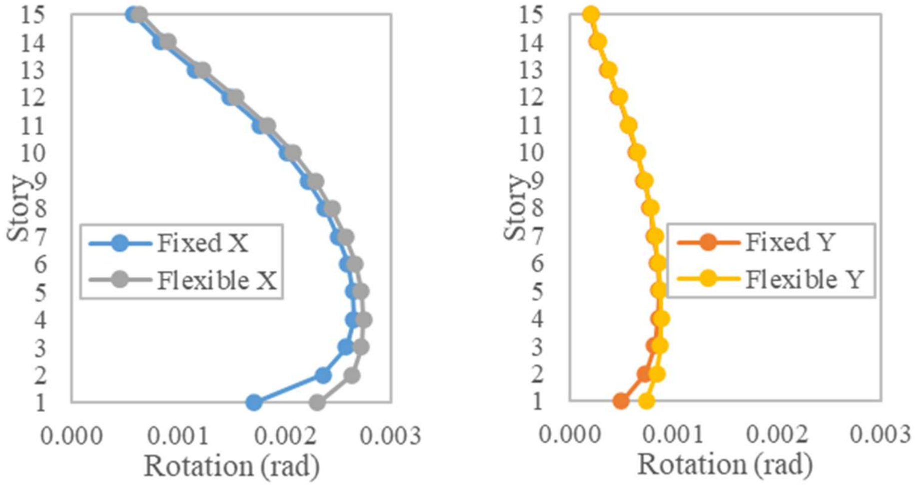
The joint rotation for fixed and flexible foundations in both X and Y directions with earthquake load EY are also analyzed and compared. In this case, the joints considered are the ones located in axis F-3. The joint rotation results of fixed and flexible foundations in X and Y directions are as follows.

Table 5.103 Joint Rotation in Axis F-3 with Fixed and Flexible Foundations in X and Y Directions

Story	Joint Rotation (rad)			
	X Direction		Y Direction	
	Fixed	Flexible	Fixed	Flexible
15	0.00058	0.00064	0.00020	0.00021
14	0.00084	0.00090	0.00027	0.00028
13	0.00116	0.00122	0.00037	0.00038
12	0.00148	0.00155	0.00047	0.00048
11	0.00177	0.00183	0.00057	0.00057
10	0.00201	0.00208	0.00065	0.00066
9	0.00222	0.00228	0.00072	0.00072
8	0.00238	0.00244	0.00077	0.00078

7	0.00250	0.00257	0.00081	0.00082
6	0.00259	0.00266	0.00084	0.00086
5	0.00264	0.00271	0.00086	0.00088
4	0.00264	0.00274	0.00086	0.00089
3	0.00258	0.00272	0.00082	0.00088
2	0.00235	0.00263	0.00073	0.00084
1	0.00170	0.00230	0.00050	0.00074

The comparison diagrams between fixed and flexible foundations generated from the joint rotation data in both X and Y directions are as follows.



(a) (b)

Figure 5.43 Joint Rotation Comparison Diagram in Axis F-3 with Fixed and Flexible Foundations in (a) X Direction and (b) Y Direction

It can be seen from the graph/diagram that the joint rotation in axis F-3 with a flexible foundation in each direction is slightly higher than the fixed counterpart in both directions.

To confirm that flexible foundations create a larger joint rotation value compared to fixed foundations, the maximum joint rotation values of both X and Y directions are analyzed and compared. The comparison is as follows.

Table 5.104 Joint Rotation Comparison Between Fixed and Flexible Foundation

Parameters	Structural Systems		Remark	Difference
	Fixed Base	Flexible Base		
Max Joint Rotation (rad)				
X Direction (Rx)	0.00306	0.00316	(+)	3.27%

Y Direction (Ry)	0.00310	0.00318	(+)	2.58%
Remark: (+) Increase, (–) Decrease, (=) Equal, (.) Unclear				

Hence, it is proven that flexible foundations create a larger joint rotation value compared to fixed foundations.

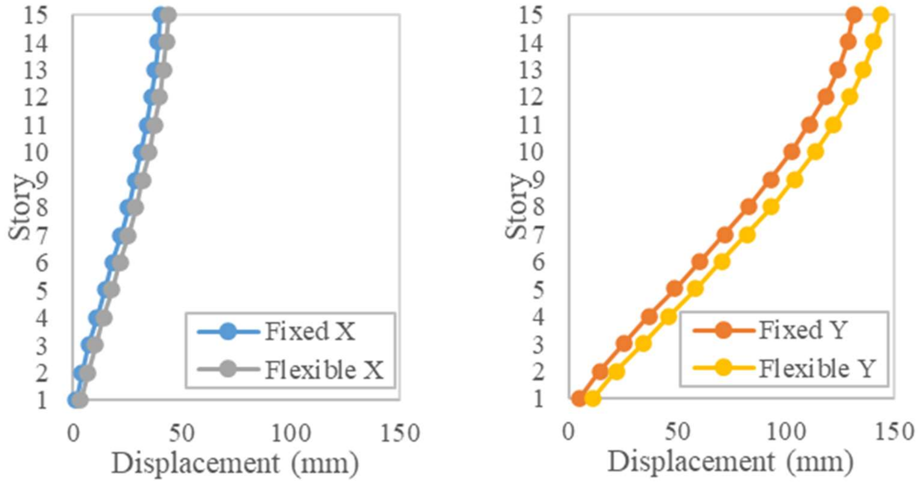
5.9.5 Horizontal Joint Displacement

The horizontal joint displacement for fixed and flexible foundations in both X and Y directions with earthquake load EY are also analyzed and compared. The horizontal joint displacement results of both fixed and flexible foundations in X and Y directions are as follows.

Table 5.105 Horizontal Joint Displacement with Fixed and Flexible Foundations in X and Y Directions

Story	Joint Displacement (mm)			
	X Direction		Y Direction	
	Fixed	Flexible	Fixed	Flexible
15	40.015	43.682	131.392	143.815
14	39.064	42.702	128.346	140.490
13	37.731	41.340	123.958	135.825
12	35.957	39.536	118.089	129.677
11	33.757	37.307	110.839	122.148
10	31.177	34.696	102.388	113.417
9	28.272	31.761	92.931	103.679
8	25.101	28.557	82.660	93.124
7	21.719	25.139	71.751	81.928
6	18.181	21.559	60.372	70.252
5	14.545	17.865	48.680	58.245
4	10.880	14.107	36.851	46.049
3	7.287	10.347	25.131	33.829
2	3.952	6.674	14.002	21.839
1	1.252	3.258	4.615	10.609

The comparison diagrams between fixed and flexible foundations generated from the horizontal joint displacement data in both X and Y directions are as follows.



(a) (b)

Figure 5.44 Horizontal Joint Displacement Comparison Diagram with Fixed and Flexible Foundations in (a) X Direction and (b) Y Direction

It can be seen from the graph/diagram that the horizontal joint displacement with a flexible foundation in each direction is slightly higher than the fixed counterpart in both directions.

To confirm that flexible foundations create a larger horizontal joint displacement value compared to fixed foundations, the maximum horizontal joint displacement of both X and Y directions are analyzed and compared. The comparison is as follows.

Table 5.106 Horizontal Joint Displacement Comparison Between Fixed and Flexible Foundation

Parameters	Structural Systems		Remark	Difference
	Fixed Base	Flexible Base		
Max Horizontal Joint Displacement (mm)				
X Direction (Ux)	40.02	43.68	(+)	9.16%
Y Direction (Uy)	131.39	143.82	(+)	9.45%
Remark: (+) Increase, (–) Decrease, (=) Equal, (.) Unclear				

Hence, it is proven that flexible foundations create a larger horizontal joint displacement value compared to fixed foundations.

5.11 Comparison Analysis

The analysis of the internal forces of a fixed base is compared with a flexible base (homogeneous soil profile) to figure out the effect of using a flexible base. The denominator used in this analysis to calculate the percentage of difference is the value of the fixed base. The comparison can be seen in the following table.

Table 5.107 Internal Forces Comparison of Fixed and Flexible Base

No	Parameters	Structural Systems		Remark	Difference
		Fixed Base	Flexible Base		
1	Fundamental Period T (s)	2.445	2.602	(+)	6.39%
2	RSA Base Shear Force (kN)				
	X Direction	12445.10	12183.30	(–)	2.10%
	Y Direction	12591.01	12292.26	(–)	2.37%
3	Max Beam Shear Force (kN)				
	X Direction	440.74	435.13	(–)	1.27%
	Y Direction	428.29	436.23	(+)	1.85%
4	Max Column Shear Force (kN)				
	X Direction	425.66	432.78	(+)	1.67%
	Y Direction	203.60	215.97	(+)	6.08%
5	Max Beam Bending Moment (kN-m)				
	X Direction	423.64	433.00	(+)	2.21%
	Y Direction	532.84	552.70	(+)	3.73%
6	Max Column Bending Moment (kN-m)				
	X Direction	1940.92	1229.76	(–)	36.64%
	Y Direction	638.13	505.15	(–)	20.84%
7	Drift Ratio (%)				
	X Direction	0.50	0.52	(+)	2.59%
	Y Direction	1.63	1.68	(+)	3.31%
8	Max Joint Rotation (rad)				
	X Direction (Rx)	0.00306	0.00316	(+)	3.27%
	Y Direction (Ry)	0.00310	0.00318	(+)	2.58%
9	Max Horizontal Joint Displacement (mm)				
	X Direction (Ux)	40.02	43.68	(+)	9.16%
	Y Direction (Uy)	131.39	143.82	(+)	9.45%
Remark: (+) Increase, (–) Decrease, (=) Equal, (.) Unclear					

From the table above, it is found that with the exception of RSA base shear force, beam shear force in X direction and column bending moment in both directions, the internal forces of the structure with flexible base provide a larger value compared to the fixed base. It can also be seen from the table that the difference between the values of column bending moment with fixed and flexible foundations in both directions is wider compared to the other parameters, with the difference of column bending moment in X direction 36.64% and in Y direction 20.84%. There are several possible factors as to why this happens, such as:

1. Fixed (rigid) foundation may resist moments in lower stories more than flexible (semi-rigid) foundation, which allows the flexible support to accommodate joint rotation better compared to fixed support.
2. The proportion of the column member dimension is rather distinctive to the proportion of the beam member dimension, with the dimension of the column being much larger than the beams.

However, as mentioned previously in Subchapter 5.9.2, the biggest difference is shown in the lower stories, while the upper stories do not show a very distinctive difference between the column bending moment values of fixed and flexible base.

When taking into consideration the building plan, albeit considered regular and symmetrical, the length of the X axis (72 m) perimeter is far greater than the Y axis (30 m). Logically, this means that the Y axis is more critical when subjected to earthquake load compared to the X axis. In this study, however, it is found that not all the parameters of the internal forces show that the Y axis is more critical. Moreover, the values between X and Y directions are similar and do not show a massive difference. This may have been caused by several factors, such as the earthquake load direction, the proportion of the column member dimension, the torsional properties, or even the building plan itself. This suggests an extension of research which may reveal the factors of the critical axis of the building plan.

Meanwhile, the spring stiffness values of a flexible foundation with a homogeneous soil profile are also compared to the values with a parabolic soil profile. The results show that the homogeneous soil profile produces relatively higher values of spring stiffness compared to the parabolic soil profile. The internal forces between a flexible foundation with homogeneous and parabolic soil profiles are also briefly compared and analyzed, which can be seen in the following table, where the denominator used in this analysis to calculate the percentage of difference is the value of the homogeneous soil profile.

Table 5.108 Internal Forces Comparison of Flexible Base with Homogeneous and Parabolic Soil Profile

No	Parameters	Soil Profile		Remark	Difference
		Homogeneous	Parabolic		
1	Fundamental Period T (s)	2.602	2.646	(+)	1.71%
2	RSA Base Shear Force (kN)				
	X Direction	12183.30	12065.01	(–)	0.97%
	Y Direction	12292.26	12191.40	(–)	0.82%
3	Max Beam Shear Force (kN)				
	X Direction	435.13	437.84	(+)	0.62%
	Y Direction	436.23	437.93	(+)	0.39%
4	Max Column Shear Force (kN)				
	X Direction	432.78	436.57	(+)	0.87%
	Y Direction	215.97	214.69	(–)	0.59%
5	Max Beam Bending Moment (kN-m)				
	X Direction	433.00	437.98	(+)	1.15%
	Y Direction	552.70	556.86	(+)	0.75%
6	Max Column Bending Moment (kN-m)				
	X Direction	1229.76	1006.25	(–)	18.18%
	Y Direction	505.15	469.94	(–)	6.97%
7	Drift Ratio (%)				
	X Direction	0.52	0.52	(+)	1.54%
	Y Direction	1.68	1.70	(+)	1.46%
8	Max Joint Rotation (rad)				
	X Direction (Rx)	0.00316	0.00319	(+)	0.98%
	Y Direction (Ry)	0.00318	0.00322	(+)	1.29%
9	Max Horizontal Joint Displacement (mm)				
	X Direction (Ux)	43.68	44.72	(+)	2.37%
	Y Direction (Uy)	143.82	147.13	(+)	2.30%
Remark: (+) Increase, (–) Decrease, (=) Equal, (.) Unclear					

From the table above, it is found that with the exception of RSA base shear force, column shear force in Y direction and column bending moment in both directions, the internal forces of flexible support with parabolic soil profile provide a larger value compared to the homogeneous soil profile. It can also be seen from the table that the difference between the values of column bending moment with fixed and flexible foundations in both directions is wider compared to the other parameters, with the difference of column bending moment in X direction 18.18% and in Y direction 6.97%. This proves that soil profile influences the internal forces working on the building. It can also be concluded that the spring stiffness of homogeneous soil profile shows a higher value compared to parabolic soil profile, which in turn elongates the fundamental period of the building. It is suggested that the research of the effects of soil shear modulus profile may be conducted further in order to realize an extensive result.

CHAPTER VI
CONCLUSION

6.1 Conclusion

Based on the analysis that has been carried out, several conclusions have been obtained, namely as follows.

1. The flexibility of a pile foundation elongates the fundamental period of the structural model, in this case a 15-story building.
2. The internal forces acting upon flexible pile foundations show that the values of base shear and column bending moment are relatively smaller than the fixed foundation, while the beam bending moment, column shear force, drift ratio, joint rotation, and horizontal joint displacement all show a higher value.
3. The soil shear modulus distribution when assumed to be uniformly distributed along the pile length or have a homogeneous soil profile generates a higher value of spring stiffness compared to when assumed to have a parabolic soil profile.

6.2 Suggestion

Based on the conclusions above, several suggestions can be concluded to obtain more optimal results in comparative analysis between internal forces of fixed and flexible foundations under dynamic loads, such as:

1. Analyzing the damping effect on flexible pile foundations.
2. Comparing the results of analysis of other internal forces.
3. Conducting more extensive research on the effect of soil shear modulus distribution on the internal forces working on the building, whether it is homogeneous or parabolic soil profiles.
4. Conducting the research using a variation of building plans, either symmetrical or not, to obtain the critical axis of the plan when subjected to earthquake loads.

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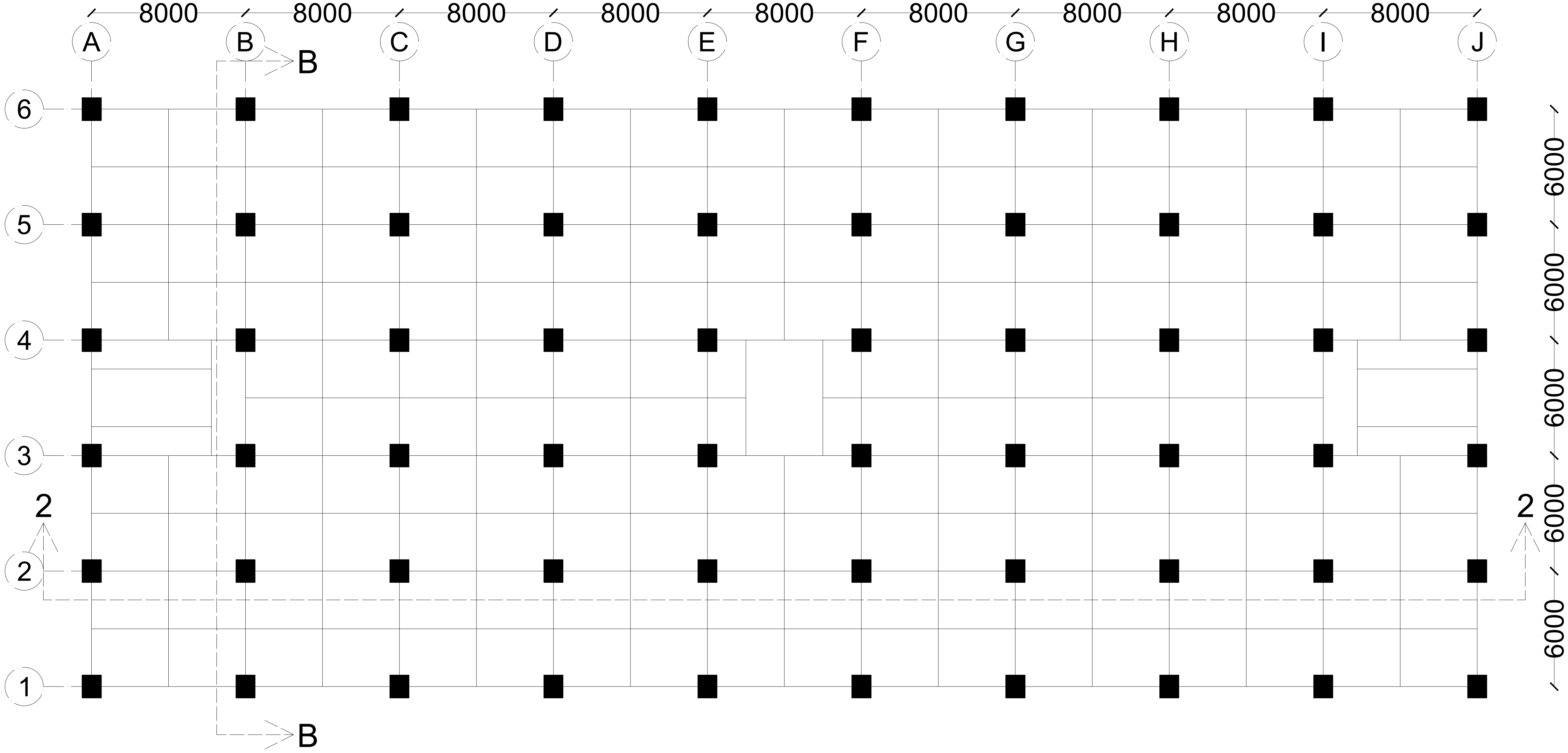
APPENDIX



FINAL PROJECT

COMPARATIVE ANALYSIS
BETWEEN INTERNAL FORCES
OF FIXED AND FLEXIBLE
FOUNDATION UNDER
DYNAMIC LOAD

DRAWING TITLE

FLOOR PLAN AXES

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APPROVED

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INFORMATION

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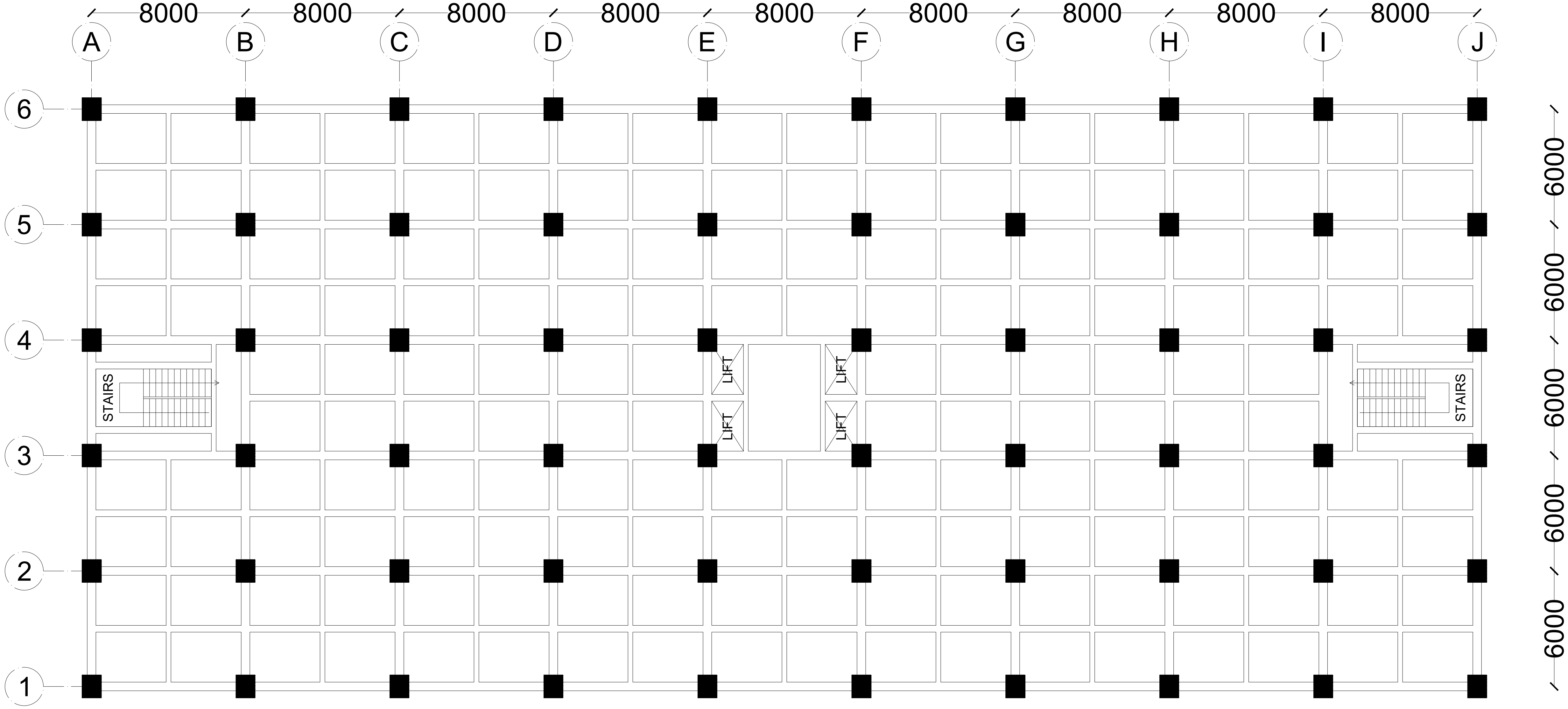

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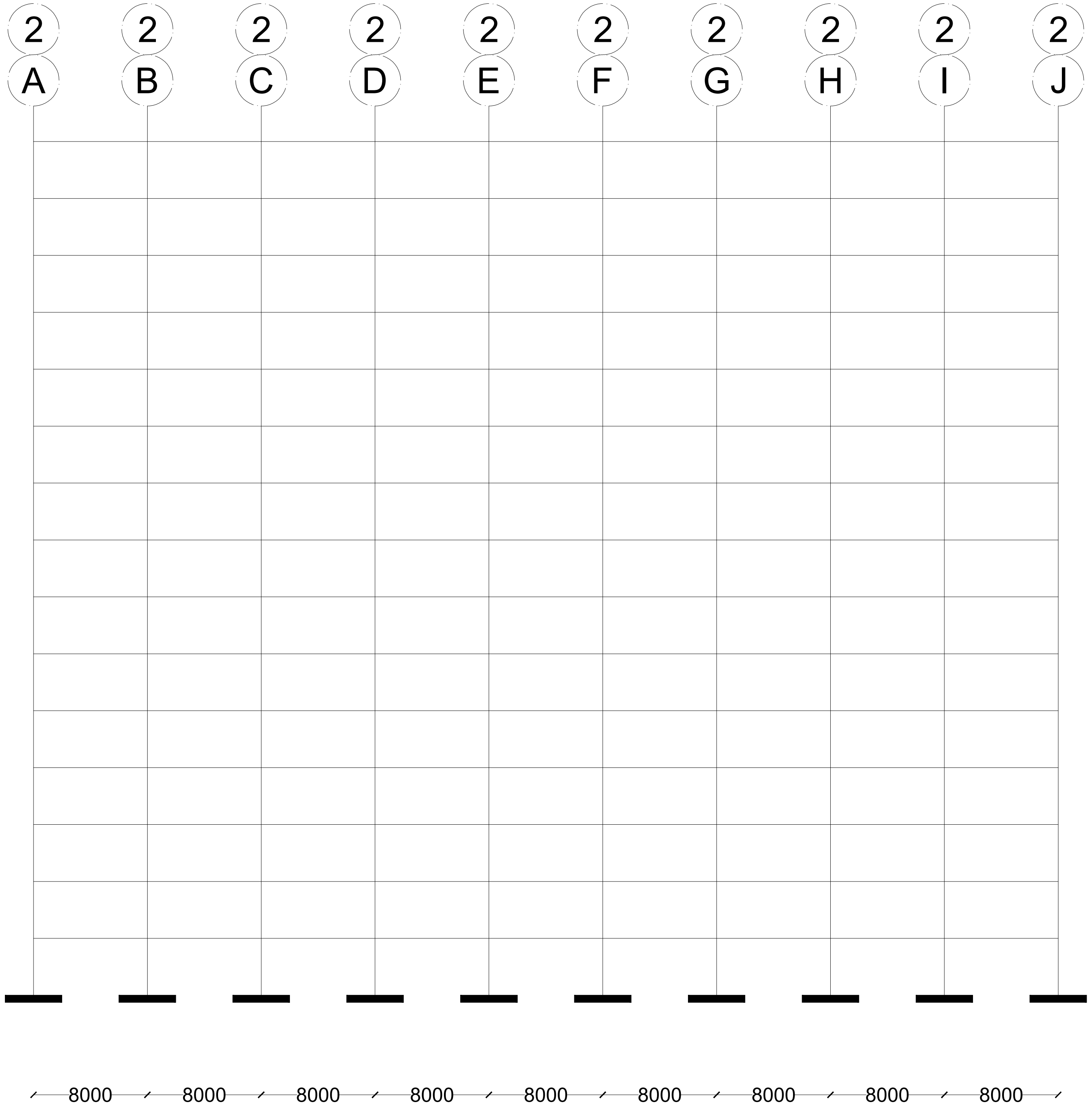

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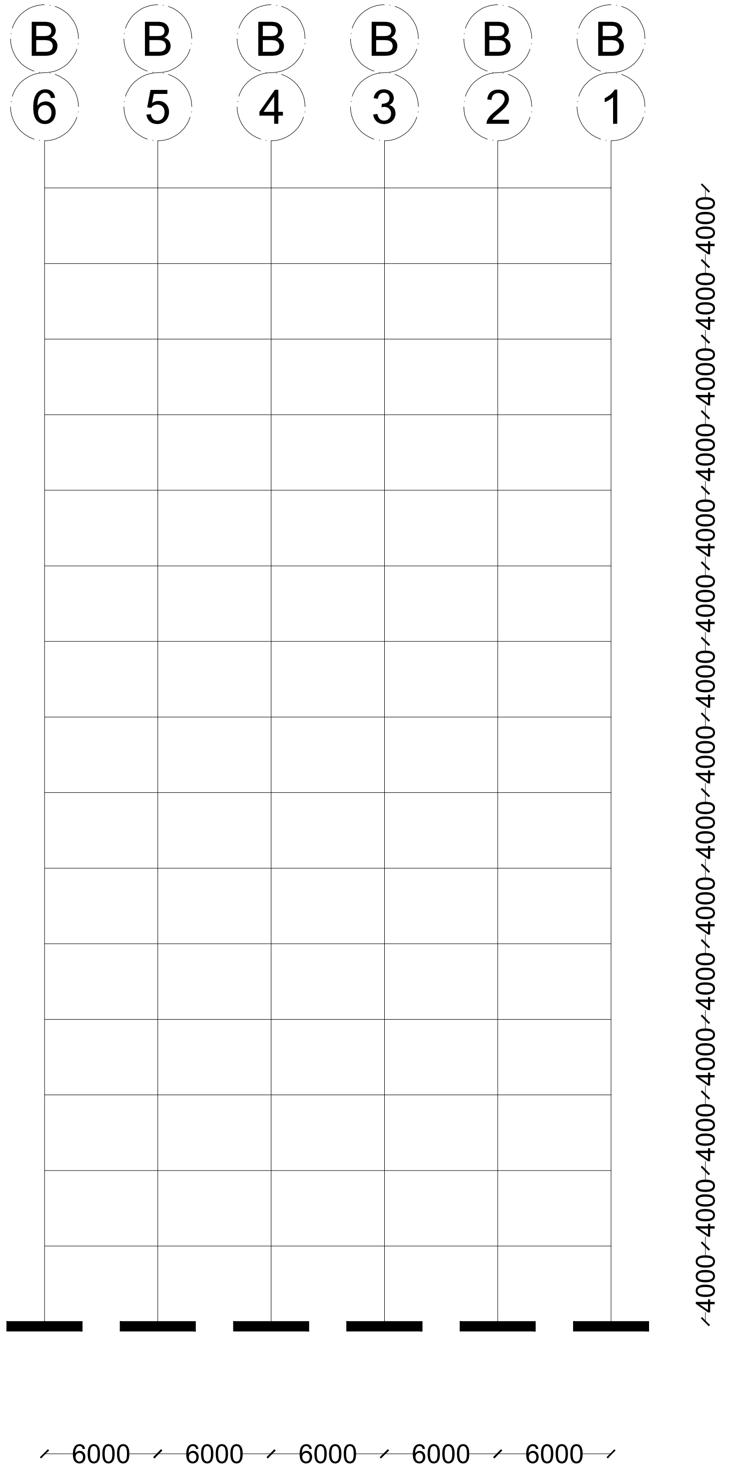

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FINAL PROJECT

COMPARATIVE ANALYSIS BETWEEN INTERNAL FORCES OF FIXED AND FLEXIBLE FOUNDATION UNDER DYNAMIC LOAD

DRAWING TITLE

CROSS-SECTIONAL VIEWS OF BUILDING WITH FIXED FOUNDATION

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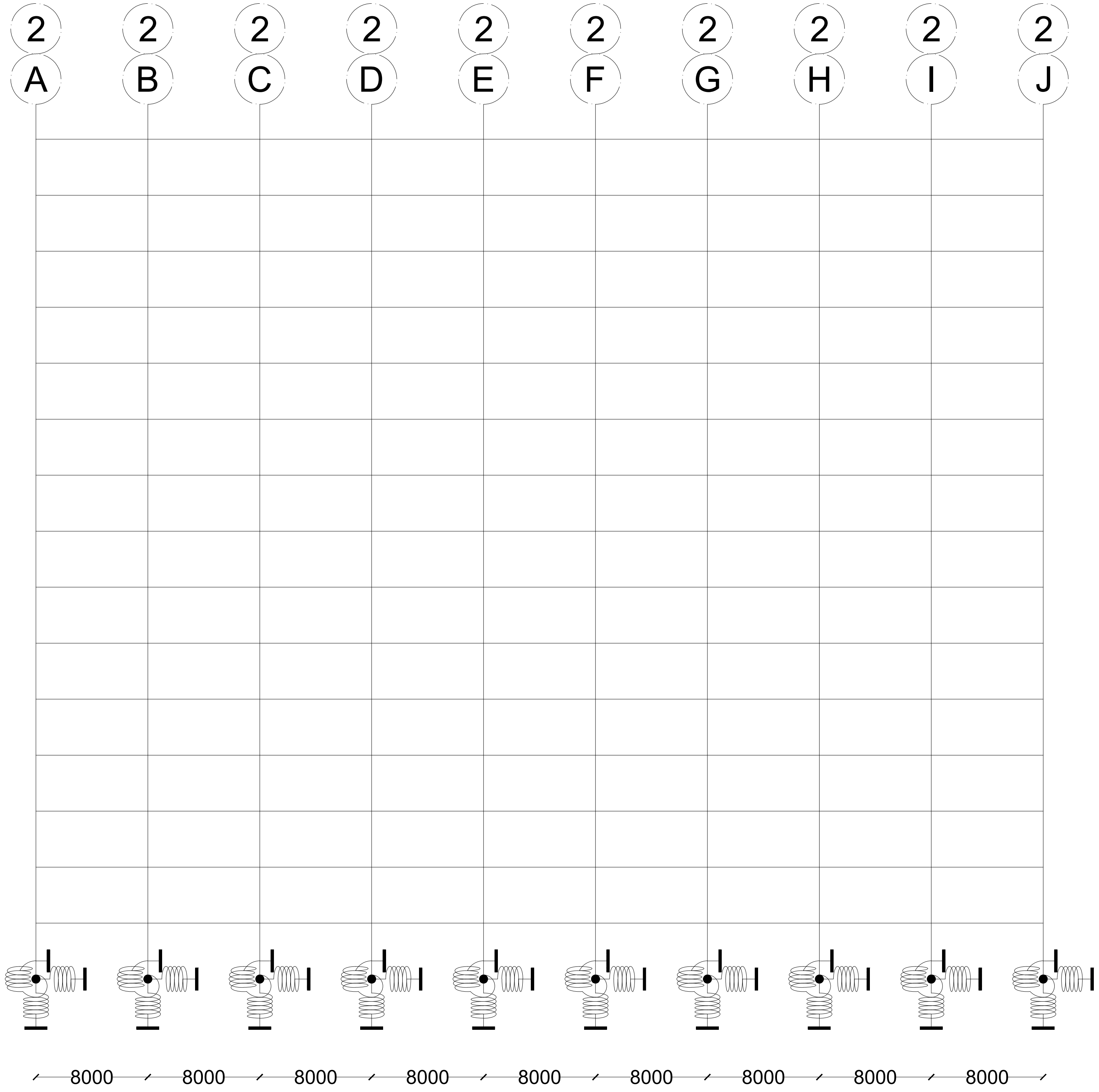

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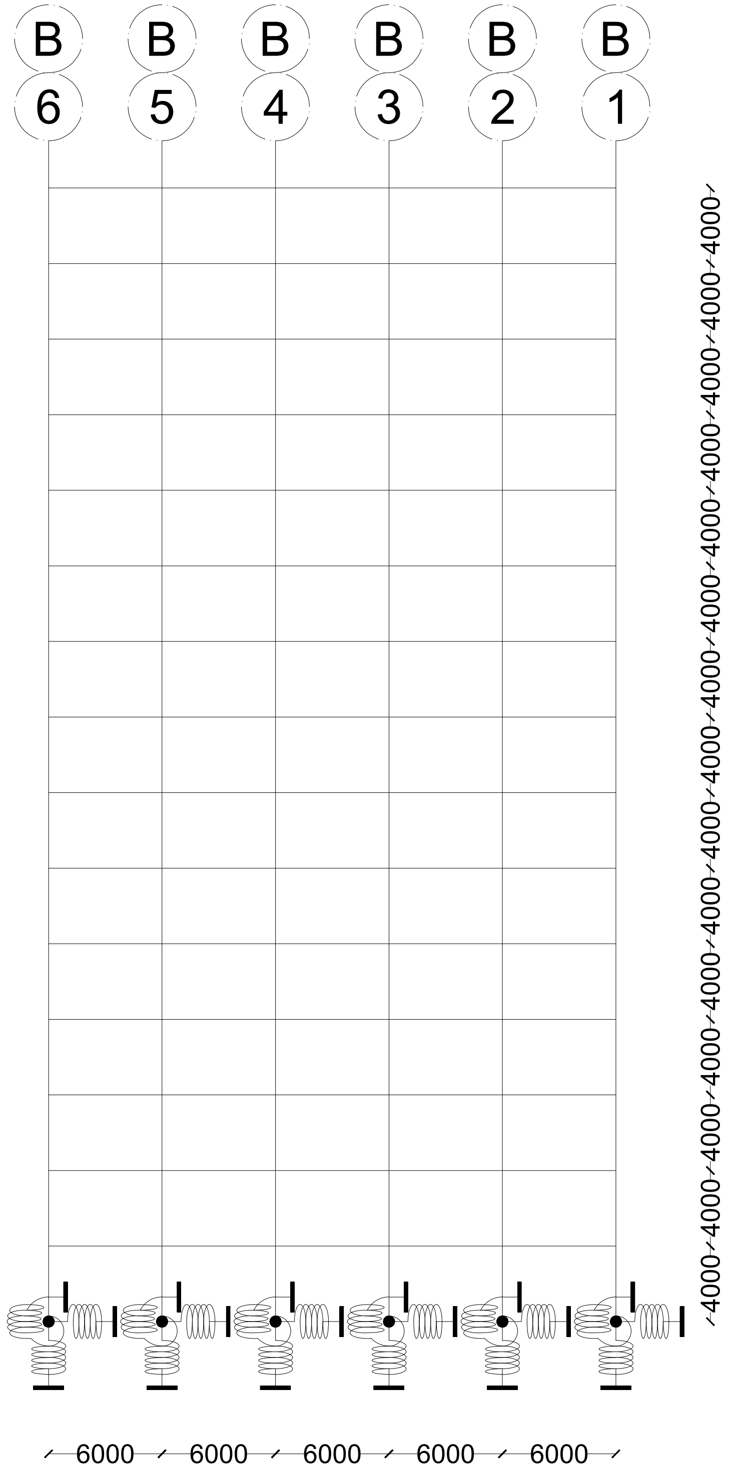

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FINAL PROJECT

COMPARATIVE ANALYSIS
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DRAWING TITLE

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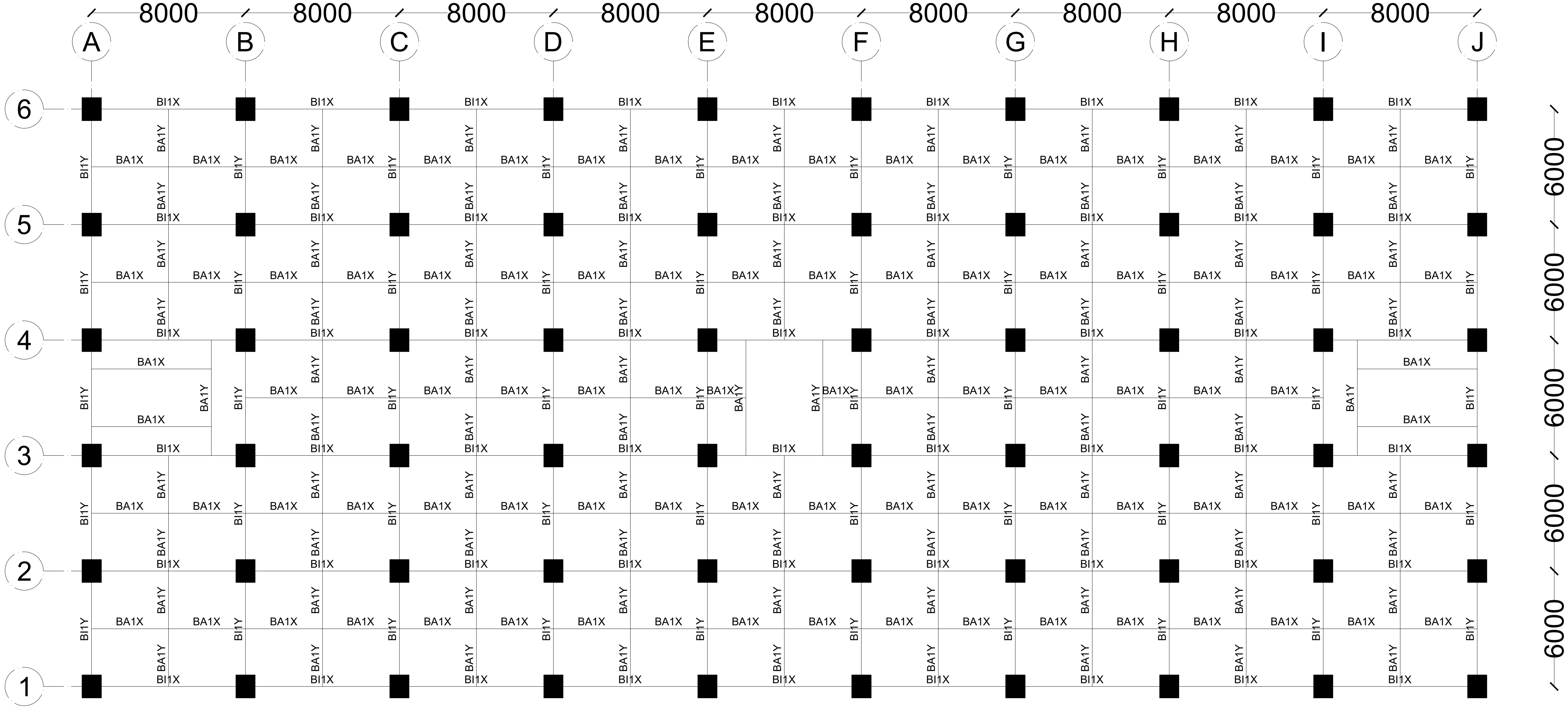

UNIVERSITAS ISLAM INDONESIA

FINAL PROJECT

COMPARATIVE ANALYSIS BETWEEN INTERNAL FORCES OF FIXED AND FLEXIBLE FOUNDATION UNDER DYNAMIC LOAD

DRAWING TITLE

BEAM CODIFICATION

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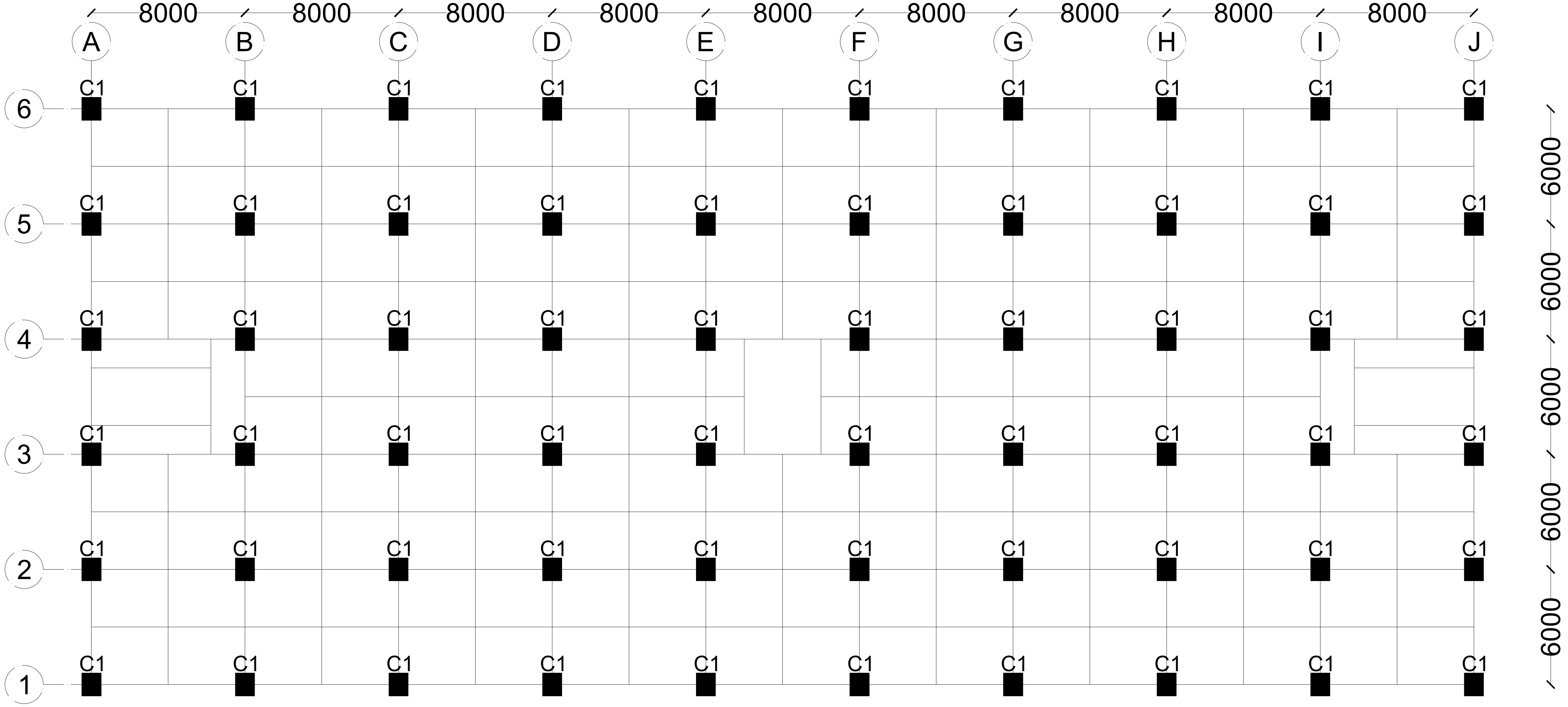

FINAL PROJECT

COMPARATIVE ANALYSIS BETWEEN INTERNAL FORCES OF FIXED AND FLEXIBLE FOUNDATION UNDER DYNAMIC LOAD

DRAWING TITLE

COLUMN CODIFICATION

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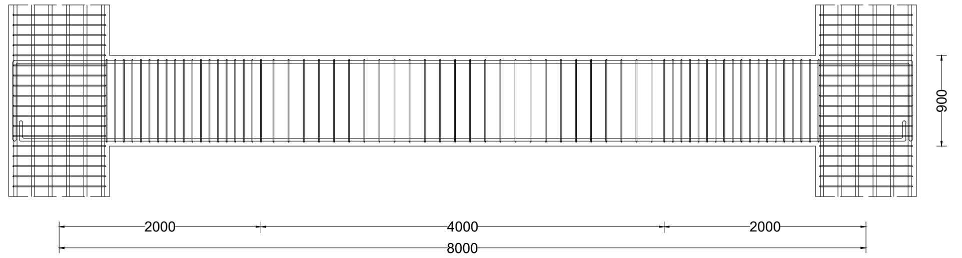
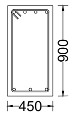
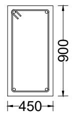
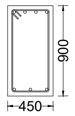
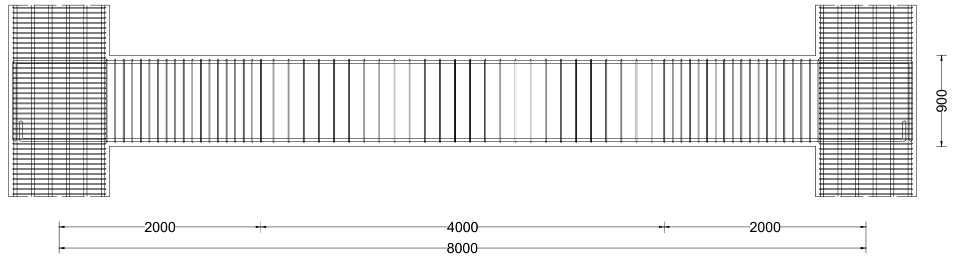
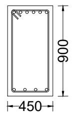
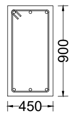
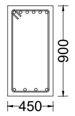
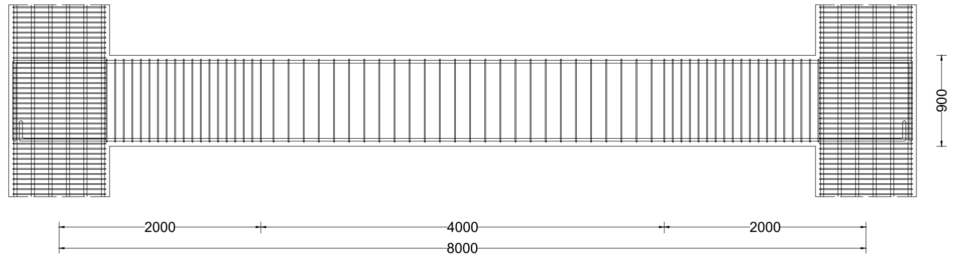
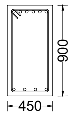
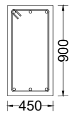
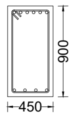
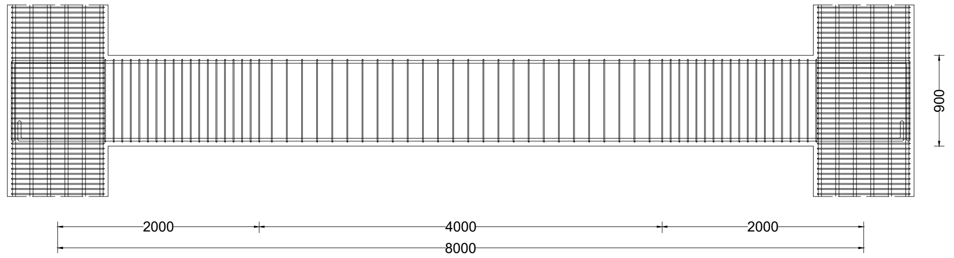
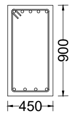
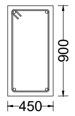
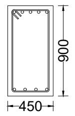
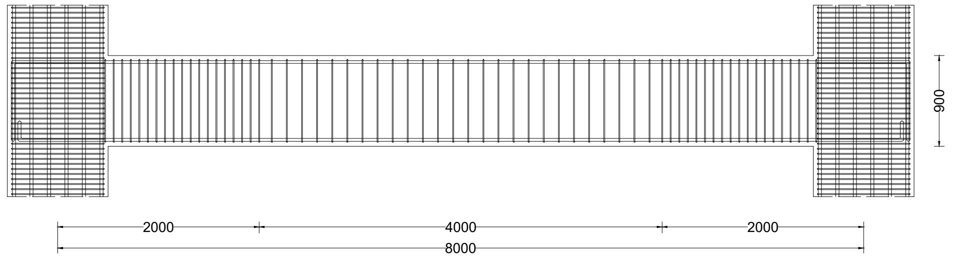
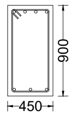
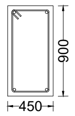
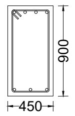
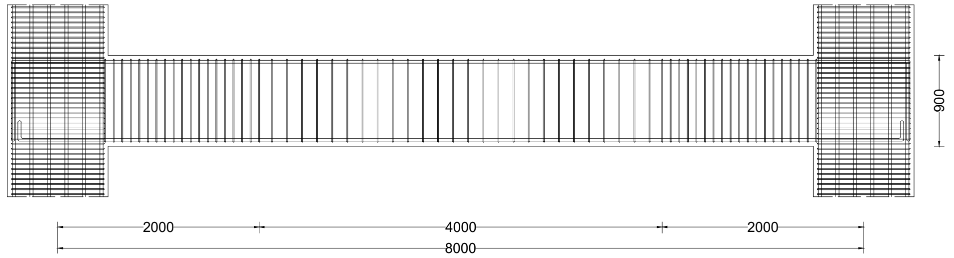
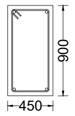
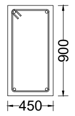
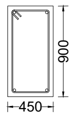
LONGITUDINAL CUT			
TRANSVERSAL CUT (CROSS-SECTION)			
UPPER REINFORCEMENT	4D28	2D28	4D28
LOWER REINFORCEMENT	3D28	2D28	3D28
SHEAR REINFORCEMENT	2P10-85 mm	2P10-150 mm	2P10-85 mm

TYPE	**BEAM BI1X STORY 2-5**		
POSITION	LEFT SUPPORT AREA	MIDDLE SPAN AREA	RIGHT SUPPORT AREA
LONGITUDINAL CUT			
TRANSVERSAL CUT (CROSS-SECTION)			
UPPER REINFORCEMENT	6D28	3D28	6D28
LOWER REINFORCEMENT	4D28	3D28	4D28
SHEAR REINFORCEMENT	2P10-85 mm	2P10-150 mm	2P10-85 mm

TYPE	**BEAM BI1X STORY 6-8**		
POSITION	LEFT SUPPORT AREA	MIDDLE SPAN AREA	RIGHT SUPPORT AREA
LONGITUDINAL CUT			
TRANSVERSAL CUT (CROSS-SECTION)			
UPPER REINFORCEMENT	6D28	3D28	6D28
LOWER REINFORCEMENT	4D28	3D28	4D28
SHEAR REINFORCEMENT	2P10-85 mm	2P10-150 mm	2P10-85 mm

TYPE	**BEAM BI1X STORY 9-11**		
POSITION	LEFT SUPPORT AREA	MIDDLE SPAN AREA	RIGHT SUPPORT AREA
LONGITUDINAL CUT			
TRANSVERSAL CUT (CROSS-SECTION)			
UPPER REINFORCEMENT	5D28	2D28	5D28
LOWER REINFORCEMENT	4D28	2D28	4D28
SHEAR REINFORCEMENT	2P10-85 mm	2P10-150 mm	2P10-85 mm

TYPE	**BEAM BI1X STORY 12-14**		
POSITION	LEFT SUPPORT AREA	MIDDLE SPAN AREA	RIGHT SUPPORT AREA
LONGITUDINAL CUT			
TRANSVERSAL CUT (CROSS-SECTION)			
UPPER REINFORCEMENT	4D28	2D28	4D28
LOWER REINFORCEMENT	3D28	2D28	3D28
SHEAR REINFORCEMENT	2P10-85 mm	2P10-150 mm	2P10-85 mm

TYPE	**BEAM BI1X STORY 15**		
POSITION	LEFT SUPPORT AREA	MIDDLE SPAN AREA	RIGHT SUPPORT AREA
LONGITUDINAL CUT			
TRANSVERSAL CUT (CROSS-SECTION)			
UPPER REINFORCEMENT	2D28	2D28	2D28
LOWER REINFORCEMENT	2D28	2D28	2D28
SHEAR REINFORCEMENT	2P10-85 mm	2P10-150 mm	2P10-85 mm

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BEAM BI1X REINFORCEMENT
DETAIL OF EVERY STORY

AUTHOR

SYAFIRA NURULITA
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APPROVED

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SUPERVISOR

INFORMATION

MILLIMETER UNIT

PAGE	SCALE
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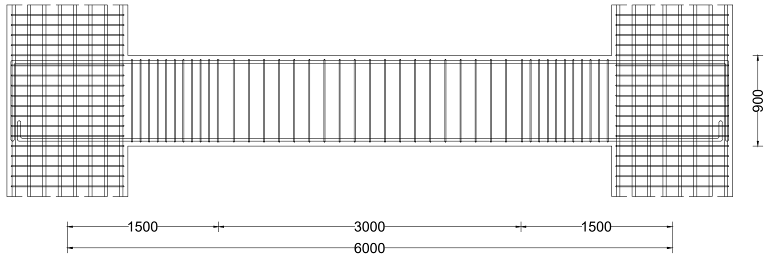
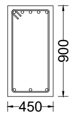
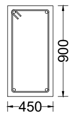
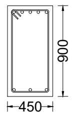
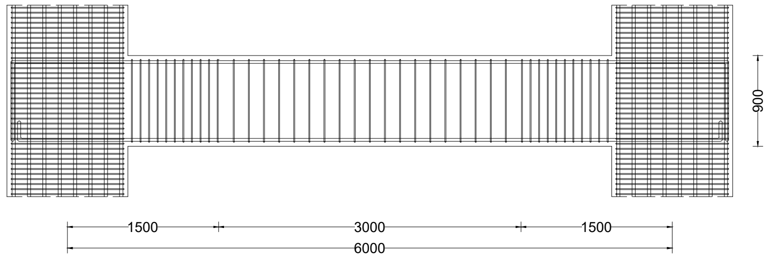
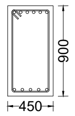
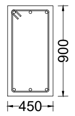
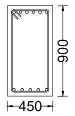
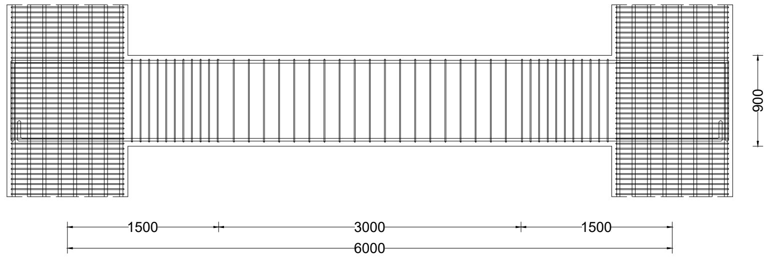
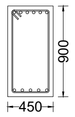
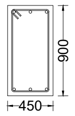
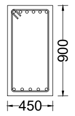
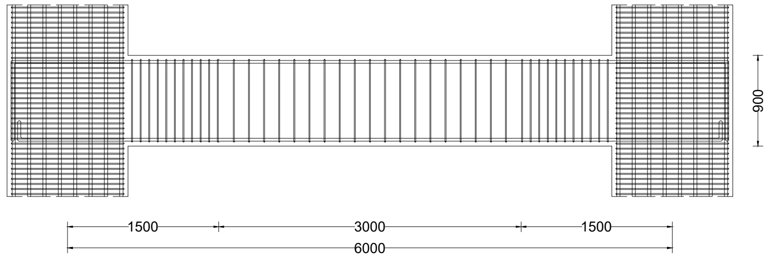
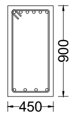
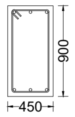
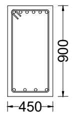
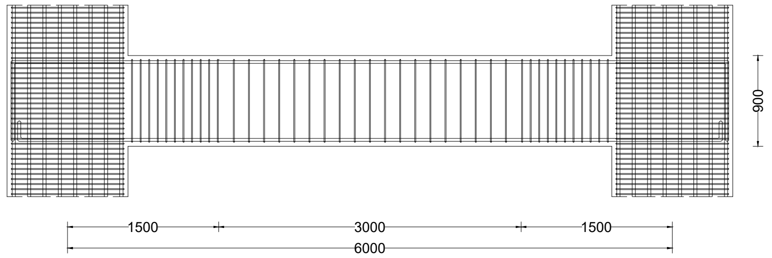
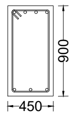
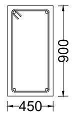
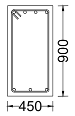
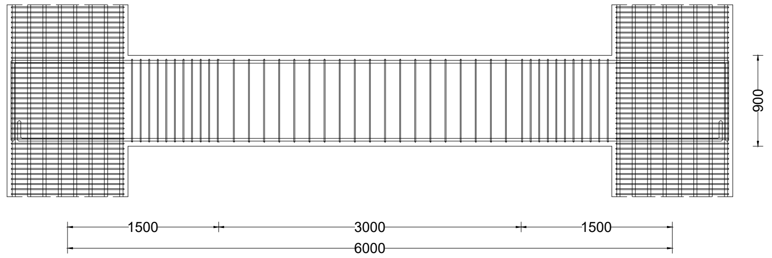
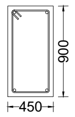
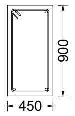
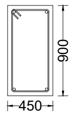
TYPE	**BEAM BI1Y STORY 1**		
POSITION	LEFT SUPPORT AREA	MIDDLE SPAN AREA	RIGHT SUPPORT AREA
LONGITUDINAL CUT			
TRANSVERSAL CUT (CROSS-SECTION)			
UPPER REINFORCEMENT	5D28	2D28	5D28
LOWER REINFORCEMENT	3D28	2D28	3D28
SHEAR REINFORCEMENT	2P10-85 mm	2P10-150 mm	2P10-85 mm

TYPE	**BEAM BI1Y STORY 2-5**		
POSITION	LEFT SUPPORT AREA	MIDDLE SPAN AREA	RIGHT SUPPORT AREA
LONGITUDINAL CUT			
TRANSVERSAL CUT (CROSS-SECTION)			
UPPER REINFORCEMENT	6D28	3D28	6D28
LOWER REINFORCEMENT	5D28	3D28	5D28
SHEAR REINFORCEMENT	2P10-85 mm	2P10-150 mm	2P10-85 mm

TYPE	**BEAM BI1Y STORY 6-8**		
POSITION	LEFT SUPPORT AREA	MIDDLE SPAN AREA	RIGHT SUPPORT AREA
LONGITUDINAL CUT			
TRANSVERSAL CUT (CROSS-SECTION)			
UPPER REINFORCEMENT	6D28	3D28	6D28
LOWER REINFORCEMENT	5D28	3D28	5D28
SHEAR REINFORCEMENT	2P10-85 mm	2P10-150 mm	2P10-85 mm

TYPE	**BEAM BI1Y STORY 9-11**		
POSITION	LEFT SUPPORT AREA	MIDDLE SPAN AREA	RIGHT SUPPORT AREA
LONGITUDINAL CUT			
TRANSVERSAL CUT (CROSS-SECTION)			
UPPER REINFORCEMENT	6D28	3D28	6D28
LOWER REINFORCEMENT	4D28	3D28	4D28
SHEAR REINFORCEMENT	2P10-85 mm	2P10-150 mm	2P10-85 mm

TYPE	**BEAM BI1Y STORY 12-14**		
POSITION	LEFT SUPPORT AREA	MIDDLE SPAN AREA	RIGHT SUPPORT AREA
LONGITUDINAL CUT			
TRANSVERSAL CUT (CROSS-SECTION)			
UPPER REINFORCEMENT	4D28	2D28	4D28
LOWER REINFORCEMENT	3D28	2D28	3D28
SHEAR REINFORCEMENT	2P10-85 mm	2P10-150 mm	2P10-85 mm

TYPE	**BEAM BI1Y STORY 15**		
POSITION	LEFT SUPPORT AREA	MIDDLE SPAN AREA	RIGHT SUPPORT AREA
LONGITUDINAL CUT			
TRANSVERSAL CUT (CROSS-SECTION)			
UPPER REINFORCEMENT	2D28	2D28	2D28
LOWER REINFORCEMENT	2D28	2D28	2D28
SHEAR REINFORCEMENT	2P10-85 mm	2P10-150 mm	2P10-85 mm

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AUTHOR

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SUPERVISOR

INFORMATION

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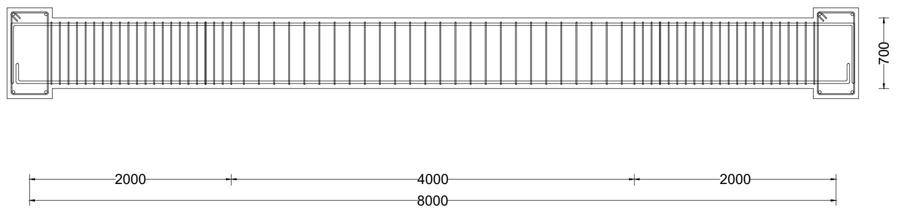
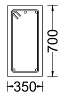
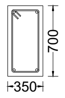
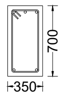
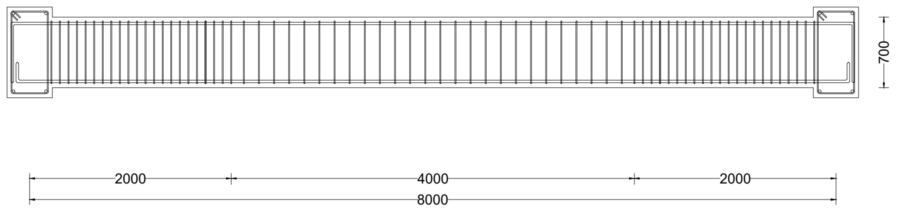
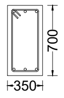
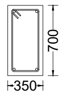
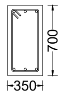
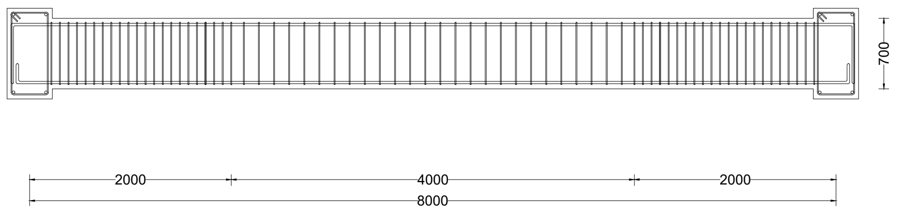
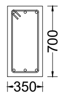
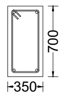
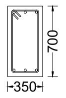
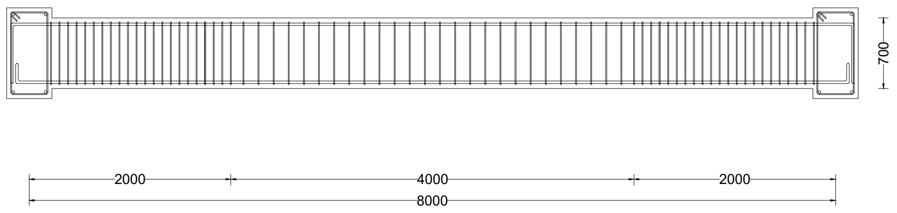
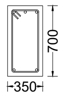
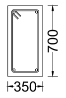
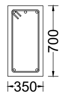
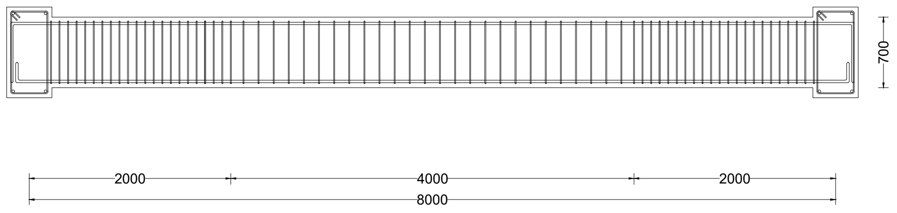
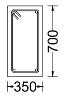
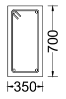
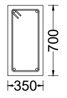
PAGE	SCALE
8	1:75

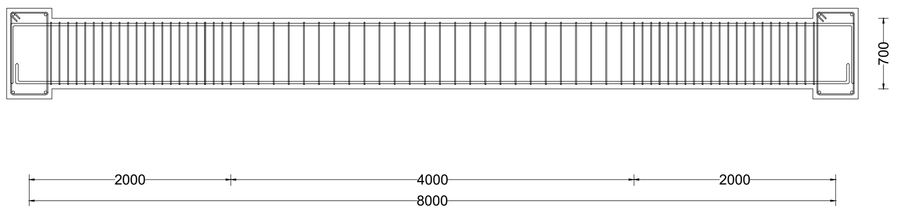
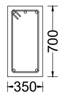
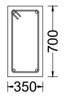
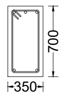
TYPE	BEAM BA1X STORY 1		
POSITION	LEFT SUPPORT AREA	MIDDLE SPAN AREA	RIGHT SUPPORT AREA
LONGITUDINAL CUT			
TRANSVERSAL CUT (CROSS-SECTION)			
UPPER REINFORCEMENT	3D25	2D25	3D25
LOWER REINFORCEMENT	2D25	2D25	2D25
SHEAR REINFORCEMENT	2P10-85 mm	2P10-150 mm	2P10-85 mm

TYPE	BEAM BA1X STORY 2-5		
POSITION	LEFT SUPPORT AREA	MIDDLE SPAN AREA	RIGHT SUPPORT AREA
LONGITUDINAL CUT			
TRANSVERSAL CUT (CROSS-SECTION)			
UPPER REINFORCEMENT	3D25	2D25	3D25
LOWER REINFORCEMENT	3D25	2D25	3D25
SHEAR REINFORCEMENT	2P10-85 mm	2P10-150 mm	2P10-85 mm

TYPE	BEAM BA1X STORY 6-8		
POSITION	LEFT SUPPORT AREA	MIDDLE SPAN AREA	RIGHT SUPPORT AREA
LONGITUDINAL CUT			
TRANSVERSAL CUT (CROSS-SECTION)			
UPPER REINFORCEMENT	3D25	2D25	3D25
LOWER REINFORCEMENT	3D25	2D25	3D25
SHEAR REINFORCEMENT	2P10-85 mm	2P10-150 mm	2P10-85 mm

TYPE	BEAM BA1X STORY 9-11		
POSITION	LEFT SUPPORT AREA	MIDDLE SPAN AREA	RIGHT SUPPORT AREA
LONGITUDINAL CUT			
TRANSVERSAL CUT (CROSS-SECTION)			
UPPER REINFORCEMENT	3D25	2D25	3D25
LOWER REINFORCEMENT	2D25	2D25	2D25
SHEAR REINFORCEMENT	2P10-85 mm	2P10-150 mm	2P10-85 mm

TYPE	BEAM BA1X STORY 12-14		
POSITION	LEFT SUPPORT AREA	MIDDLE SPAN AREA	RIGHT SUPPORT AREA
LONGITUDINAL CUT			
TRANSVERSAL CUT (CROSS-SECTION)			
UPPER REINFORCEMENT	2D25	2D25	2D25
LOWER REINFORCEMENT	2D25	2D25	2D25
SHEAR REINFORCEMENT	2P10-85 mm	2P10-150 mm	2P10-85 mm

TYPE	BEAM BA1X STORY 15		
POSITION	LEFT SUPPORT AREA	MIDDLE SPAN AREA	RIGHT SUPPORT AREA
LONGITUDINAL CUT			
TRANSVERSAL CUT (CROSS-SECTION)			
UPPER REINFORCEMENT	3D25	2D25	3D25
LOWER REINFORCEMENT	2D25	2D25	2D25
SHEAR REINFORCEMENT	2P10-85 mm	2P10-150 mm	2P10-85 mm

UNIVERSITAS ISLAM INDONESIA

FINAL PROJECT

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DRAWING TITLE

BEAM BA1X REINFORCEMENT DETAIL OF EVERY STORY

AUTHOR

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(18511126)

APPROVED

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SUPERVISOR

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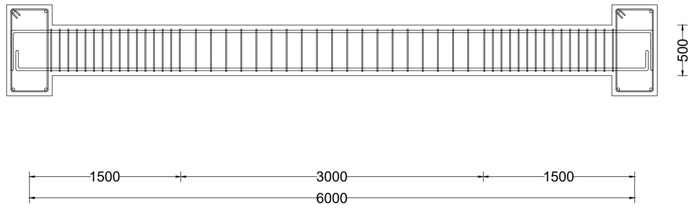
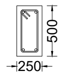
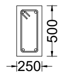
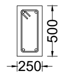
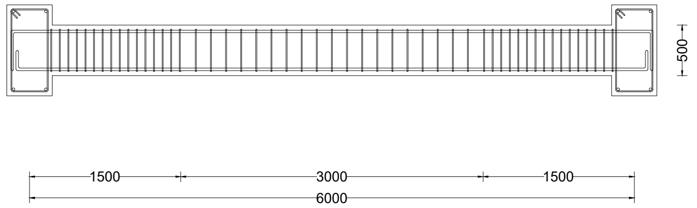
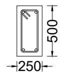
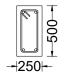
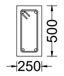
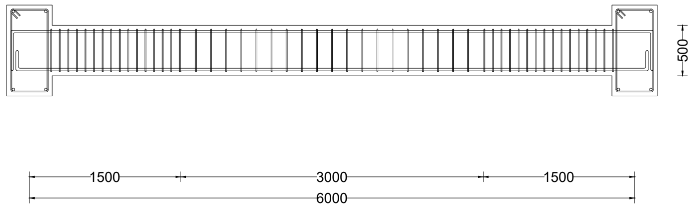
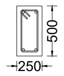
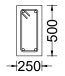
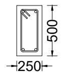
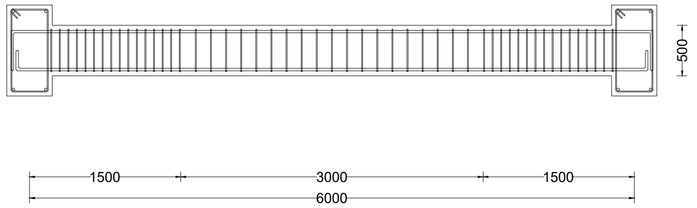
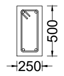
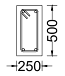
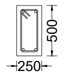
MILLIMETER UNIT

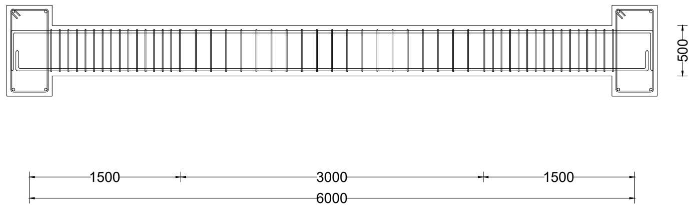
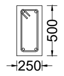
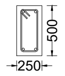
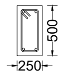
PAGE	SCALE
9	1:75

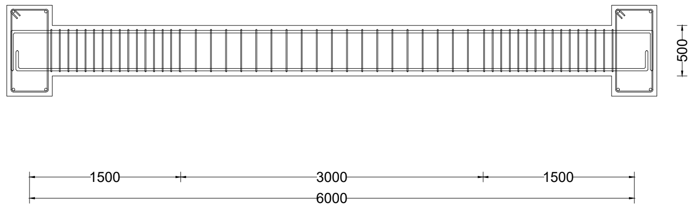
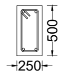
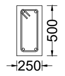
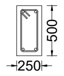
TYPE	**BEAM BA1Y STORY 1**		
POSITION	LEFT SUPPORT AREA	MIDDLE SPAN AREA	RIGHT SUPPORT AREA
LONGITUDINAL CUT			
TRANSVERSAL CUT (CROSS-SECTION)			
UPPER REINFORCEMENT	2D25	2D25	2D25
LOWER REINFORCEMENT	2D25	2D25	2D25
SHEAR REINFORCEMENT	2P10-85 mm	2P10-150 mm	2P10-85 mm

TYPE	**BEAM BA1Y STORY 9-11**		
POSITION	LEFT SUPPORT AREA	MIDDLE SPAN AREA	RIGHT SUPPORT AREA
LONGITUDINAL CUT			
TRANSVERSAL CUT (CROSS-SECTION)			
UPPER REINFORCEMENT	2D25	2D25	2D25
LOWER REINFORCEMENT	2D25	2D25	2D25
SHEAR REINFORCEMENT	2P10-85 mm	2P10-150 mm	2P10-85 mm

TYPE	**BEAM BA1Y STORY 2-5**		
POSITION	LEFT SUPPORT AREA	MIDDLE SPAN AREA	RIGHT SUPPORT AREA
LONGITUDINAL CUT			
TRANSVERSAL CUT (CROSS-SECTION)			
UPPER REINFORCEMENT	2D25	2D25	2D25
LOWER REINFORCEMENT	2D25	2D25	2D25
SHEAR REINFORCEMENT	2P10-85 mm	2P10-150 mm	2P10-85 mm

TYPE	**BEAM BA1Y STORY 12-14**		
POSITION	LEFT SUPPORT AREA	MIDDLE SPAN AREA	RIGHT SUPPORT AREA
LONGITUDINAL CUT			
TRANSVERSAL CUT (CROSS-SECTION)			
UPPER REINFORCEMENT	2D25	2D25	2D25
LOWER REINFORCEMENT	2D25	2D25	2D25
SHEAR REINFORCEMENT	2P10-85 mm	2P10-150 mm	2P10-85 mm

TYPE	**BEAM BA1Y STORY 6-8**		
POSITION	LEFT SUPPORT AREA	MIDDLE SPAN AREA	RIGHT SUPPORT AREA
LONGITUDINAL CUT			
TRANSVERSAL CUT (CROSS-SECTION)			
UPPER REINFORCEMENT	2D25	2D25	2D25
LOWER REINFORCEMENT	2D25	2D25	2D25
SHEAR REINFORCEMENT	2P10-85 mm	2P10-150 mm	2P10-85 mm

TYPE	**BEAM BA1Y STORY 15**		
POSITION	LEFT SUPPORT AREA	MIDDLE SPAN AREA	RIGHT SUPPORT AREA
LONGITUDINAL CUT			
TRANSVERSAL CUT (CROSS-SECTION)			
UPPER REINFORCEMENT	2D25	2D25	2D25
LOWER REINFORCEMENT	2D25	2D25	2D25
SHEAR REINFORCEMENT	2P10-85 mm	2P10-150 mm	2P10-85 mm

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DRAWING TITLE

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INFORMATION

MILLIMETER UNIT

PAGE	SCALE
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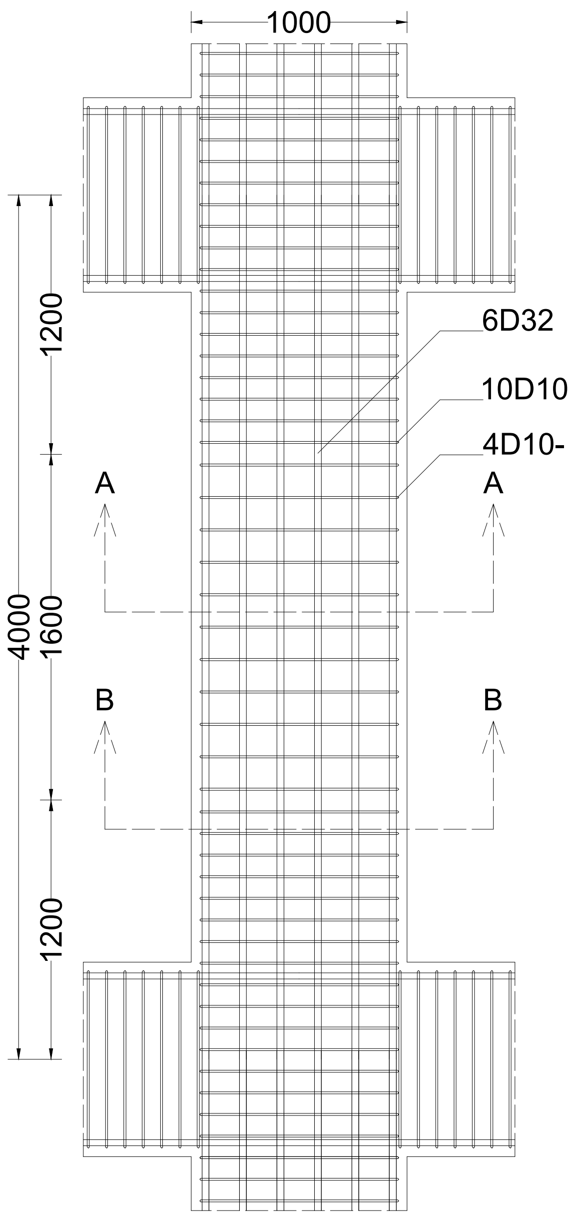
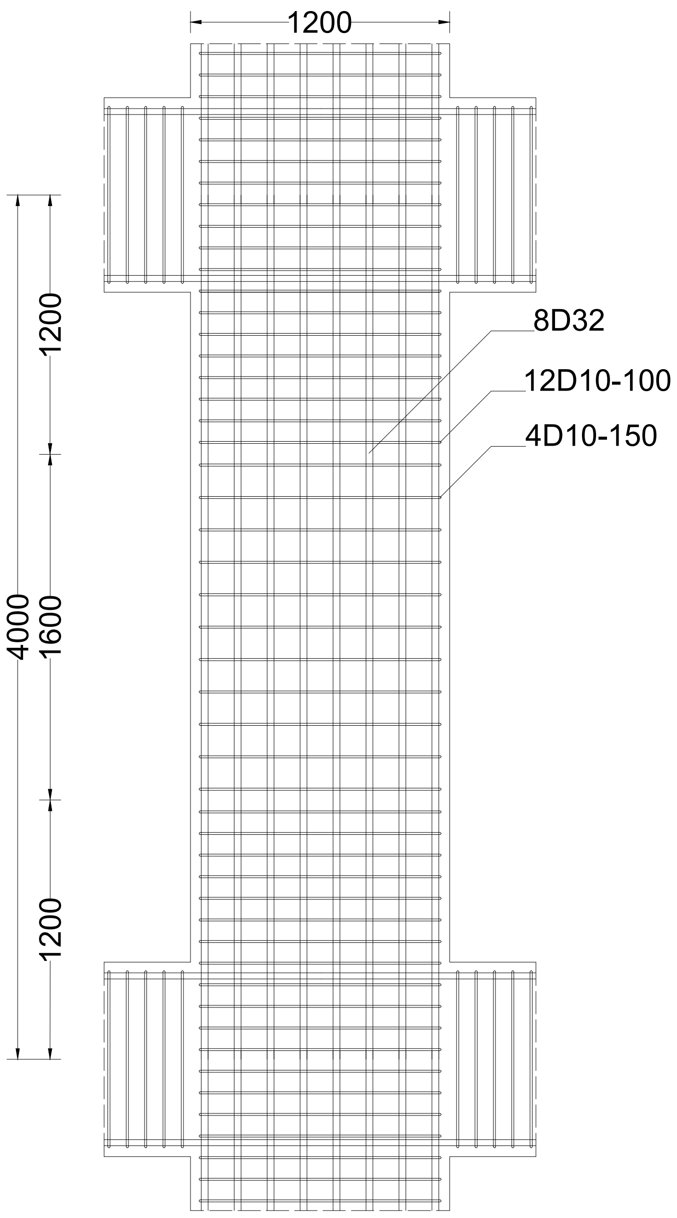


COLUMN C1 X DIRECTION
STORY 1



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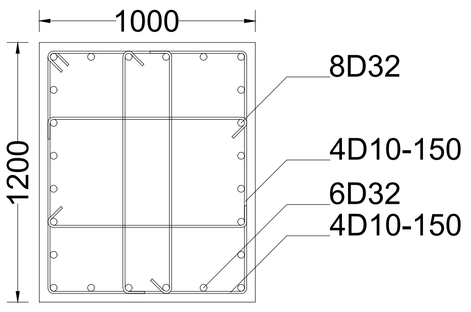
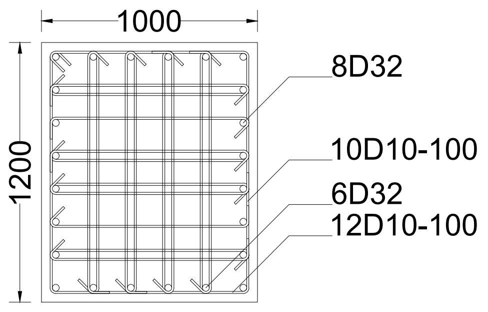
STORY 1



SECTION CUT A-A



SECTION CUT B-B

FINAL PROJECT

COMPARATIVE ANALYSIS
BETWEEN INTERNAL FORCES
OF FIXED AND FLEXIBLE
FOUNDATION UNDER
DYNAMIC LOAD

DRAWING TITLE

COLUMN C1 REINFORCEMENT
DETAIL OF STORY 1
IN X AND Y DIRECTIONS

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MILLIMETER UNIT

PAGE	SCALE
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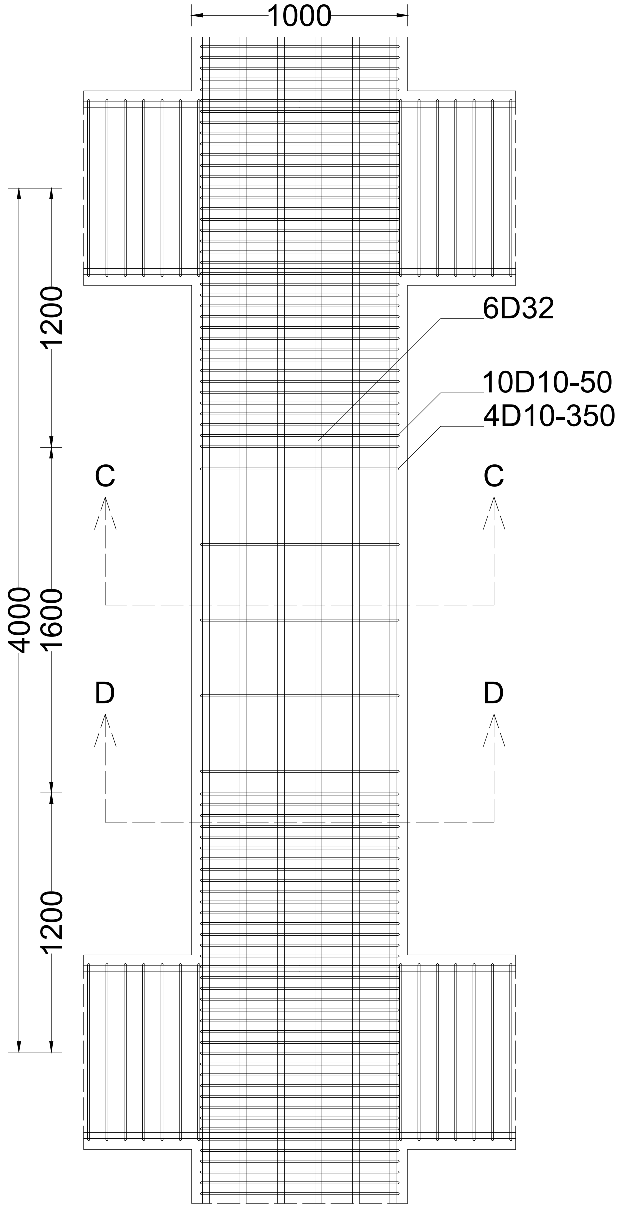
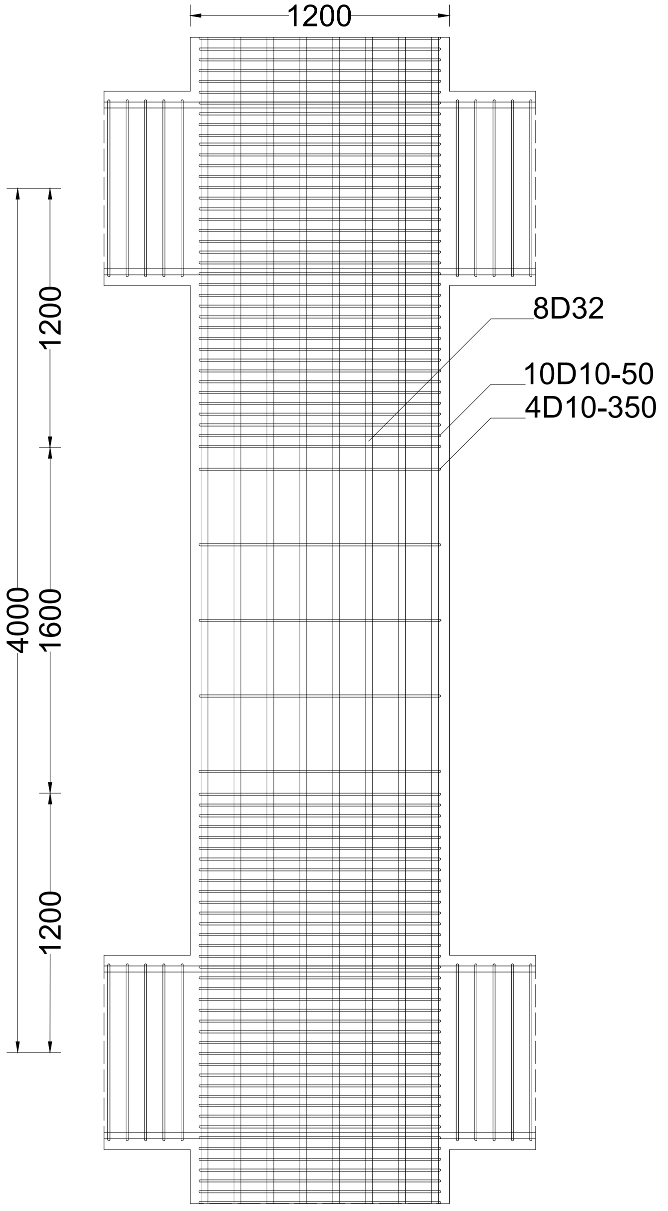


COLUMN C1 X DIRECTION
STORIES 2-15



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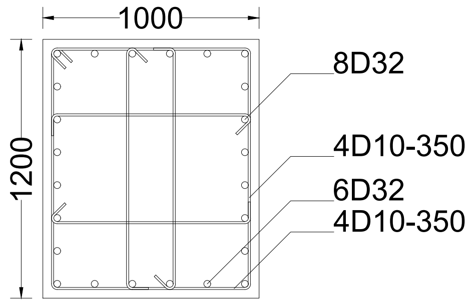
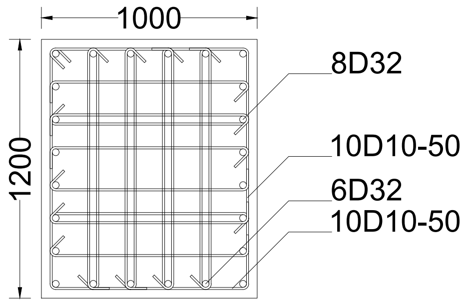
STORIES 2-15



SECTION CUT C-C



SECTION CUT D-D

UNIVERSITAS ISLAM INDONESIA

FINAL PROJECT

COMPARATIVE ANALYSIS
BETWEEN INTERNAL FORCES
OF FIXED AND FLEXIBLE
FOUNDATION UNDER
DYNAMIC LOAD

DRAWING TITLE

COLUMN C1 REINFORCEMENT
DETAIL OF STORIES 2-15
IN X AND Y DIRECTIONS

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MILLIMETER UNIT

PAGE	SCALE
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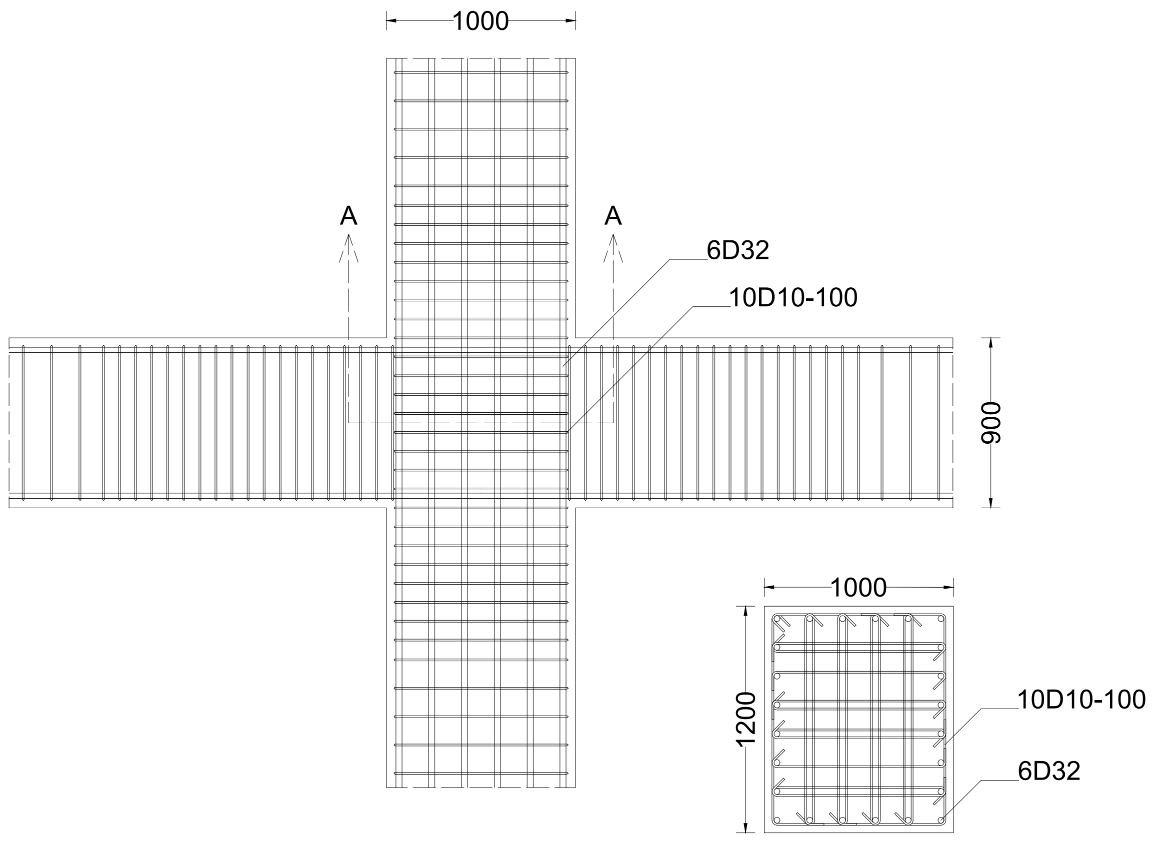
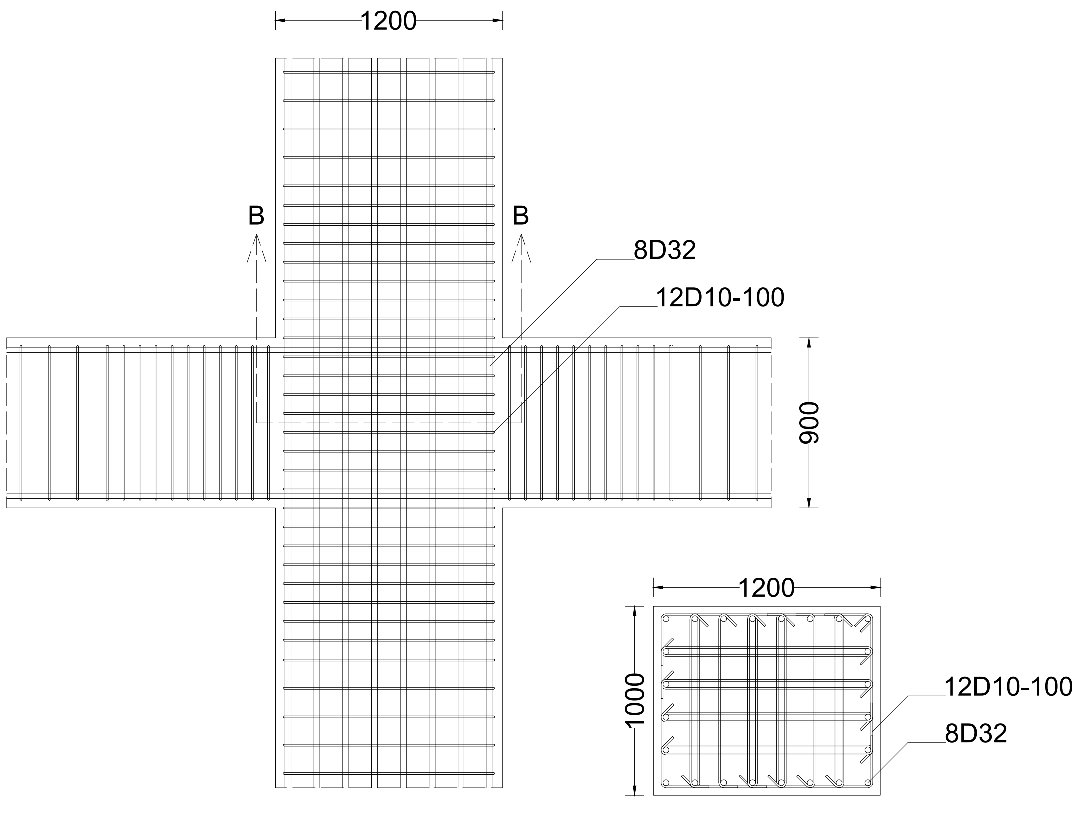


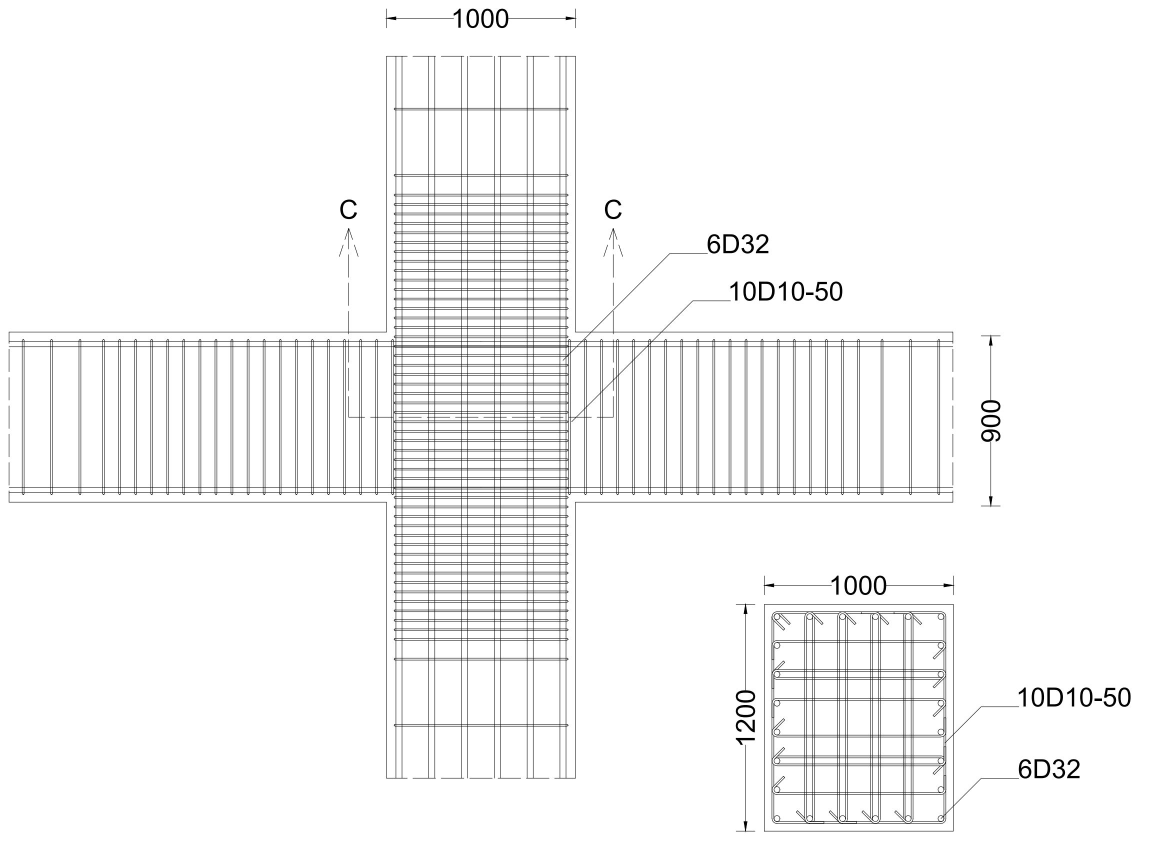
BEAM-COLUMN JOINT X DIRECTION
STORY 1

SECTION CUT A-A



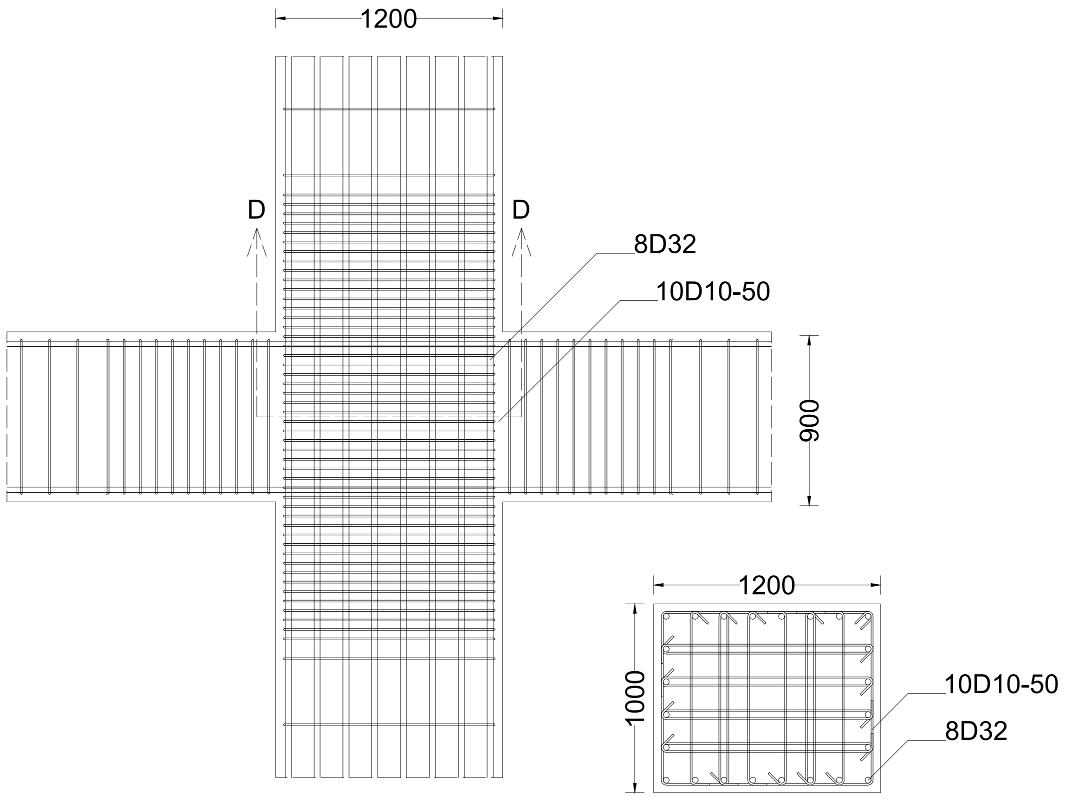
SECTION CUT B-B

BEAM-COLUMN JOINT Y DIRECTION
STORY 1



BEAM-COLUMN JOINT X DIRECTION
STORIES 2-15

SECTION CUT C-C



SECTION CUT D-D

BEAM-COLUMN JOINT Y DIRECTION
STORIES 2-15

UNIVERSITAS ISLAM INDONESIA

FINAL PROJECT

COMPARATIVE ANALYSIS
BETWEEN INTERNAL FORCES
OF FIXED AND FLEXIBLE
FOUNDATION UNDER
DYNAMIC LOAD

DRAWING TITLE

BEAM-COLUMN JOINT
REINFORCEMENT DETAIL OF
EVERY STORY

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MILLIMETER UNIT

PAGE	SCALE
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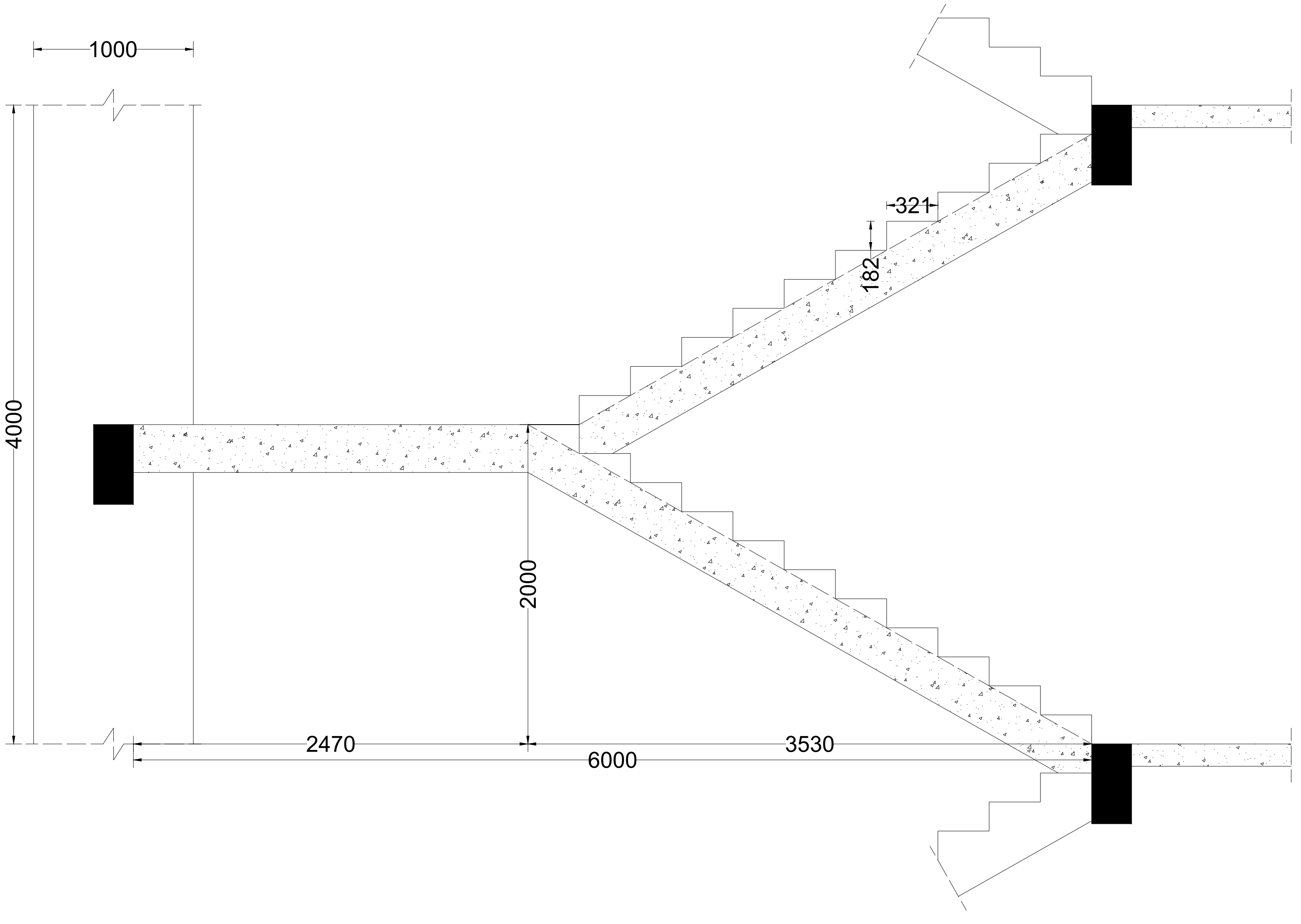
UPPER VIEW OF STAIRS



SECTION CUT A-A

UNIVERSITAS ISLAM INDONESIA

FINAL PROJECT

COMPARATIVE ANALYSIS BETWEEN INTERNAL FORCES OF FIXED AND FLEXIBLE FOUNDATION UNDER DYNAMIC LOAD

DRAWING TITLE

STAIRS UPPER VIEW AND SECTION CUT

AUTHOR

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INFORMATION

MILLIMETER UNIT

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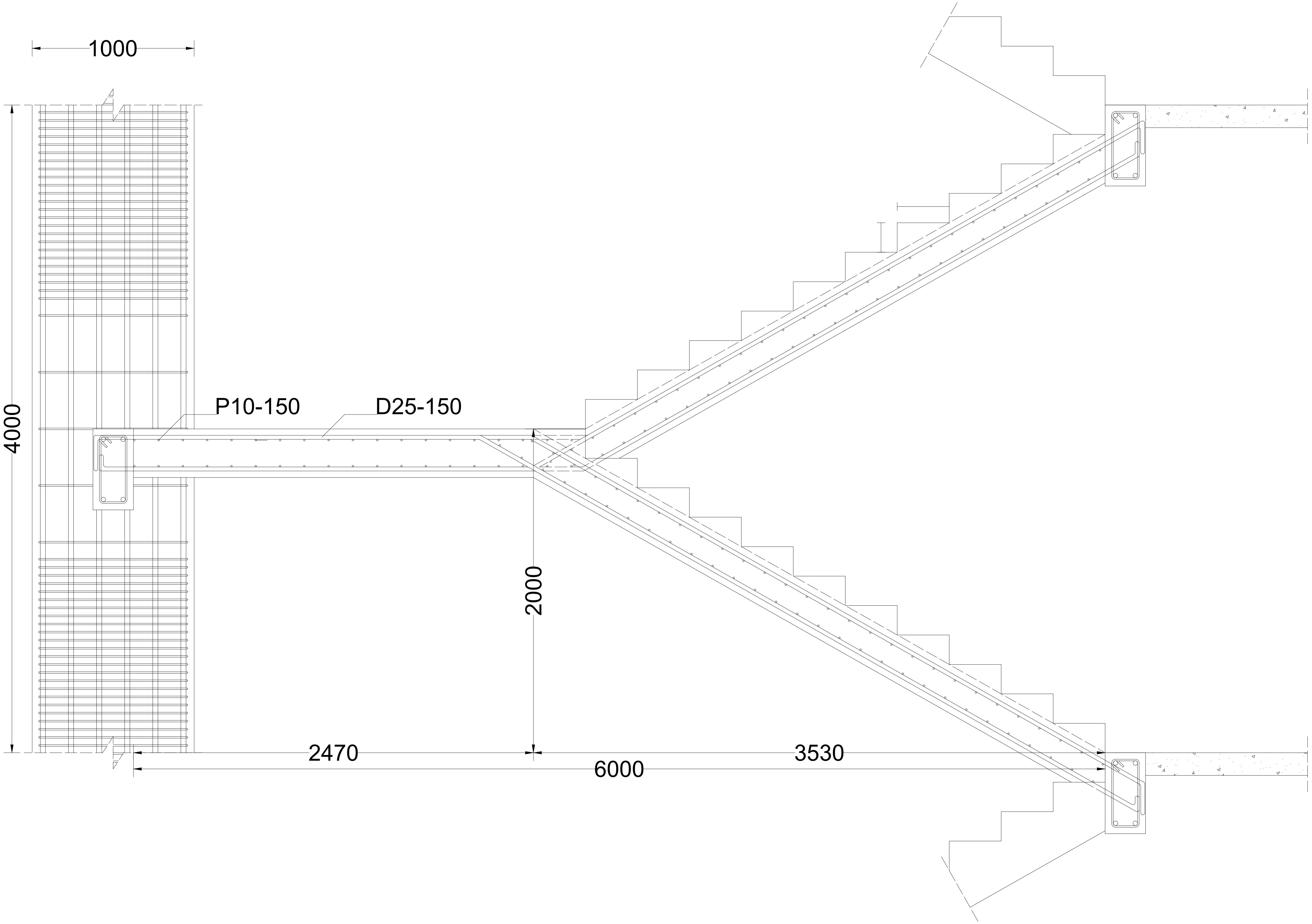

FINAL PROJECT

COMPARATIVE ANALYSIS
BETWEEN INTERNAL FORCES
OF FIXED AND FLEXIBLE
FOUNDATION UNDER
DYNAMIC LOAD

DRAWING TITLE

STAIRS REINFORCEMENT
DETAIL IN SECTION CUT A-A

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MILLIMETER UNIT

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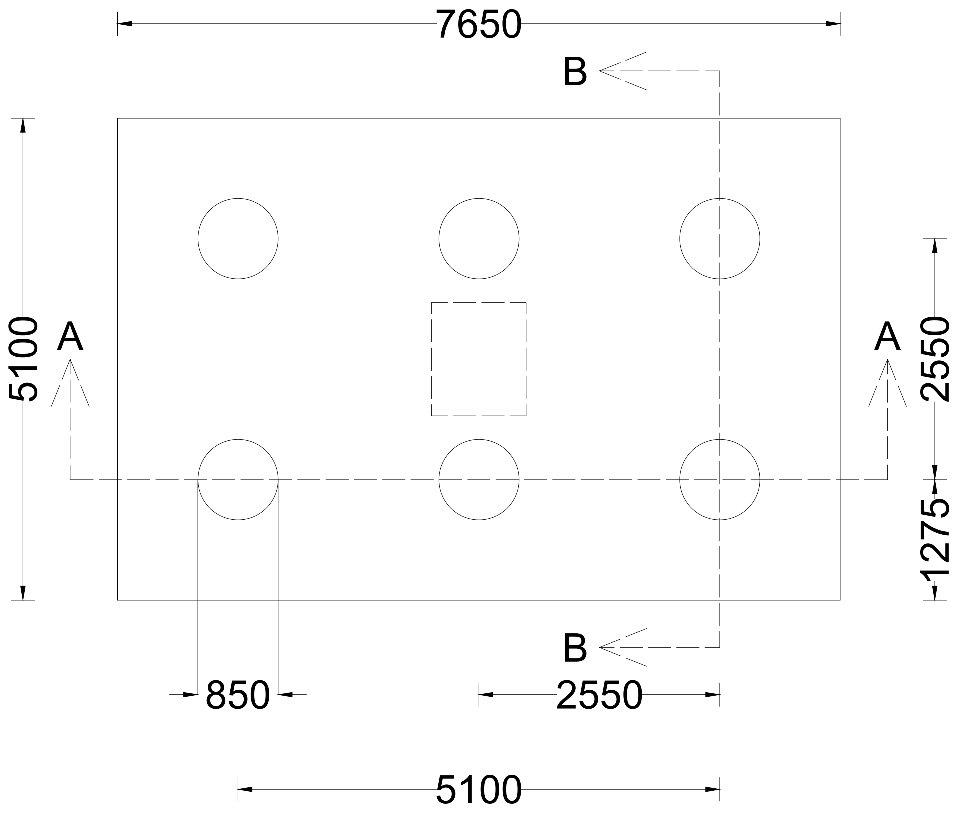
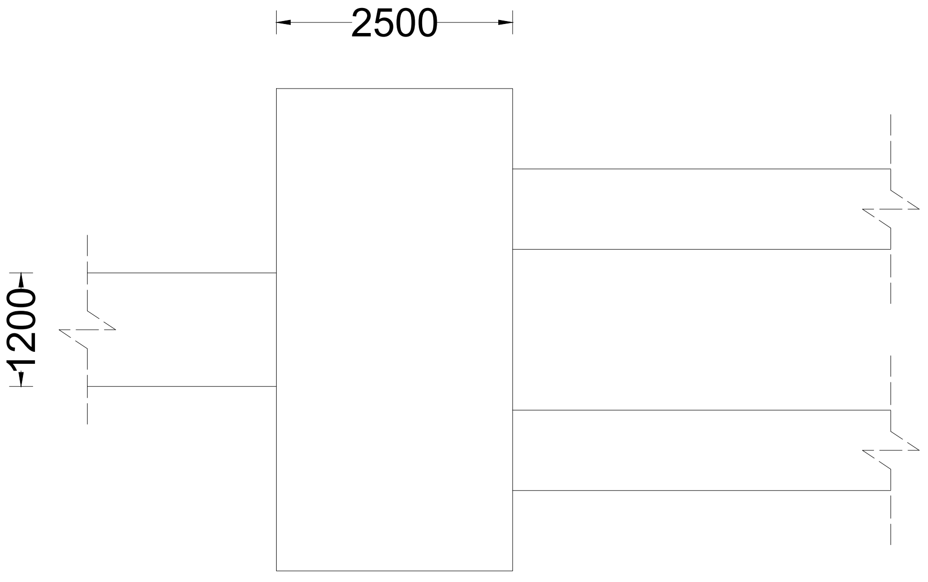
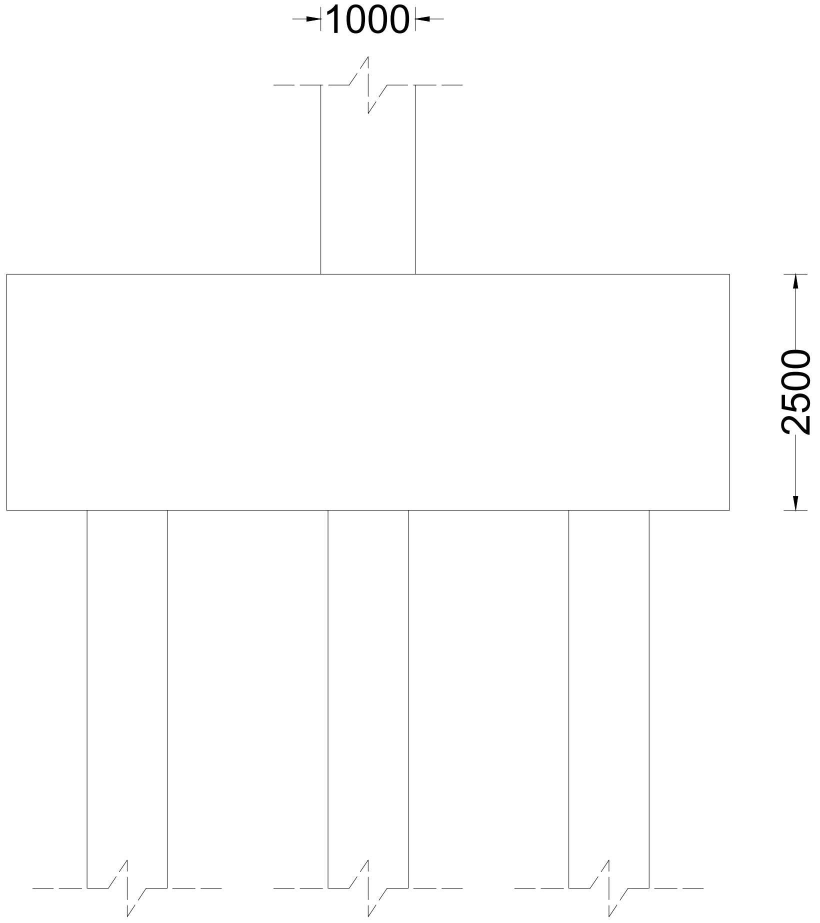




SECTION CUT B-B PROJECTION



SECTION CUT A-A PROJECTION

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FINAL PROJECT

COMPARATIVE ANALYSIS
BETWEEN INTERNAL FORCES
OF FIXED AND FLEXIBLE
FOUNDATION UNDER
DYNAMIC LOAD

DRAWING TITLE

PILE CAP DIMENSION
AND SECTION CUT
PROJECTIONS

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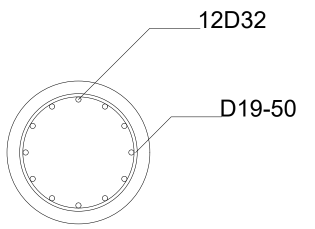

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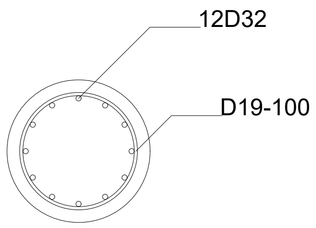

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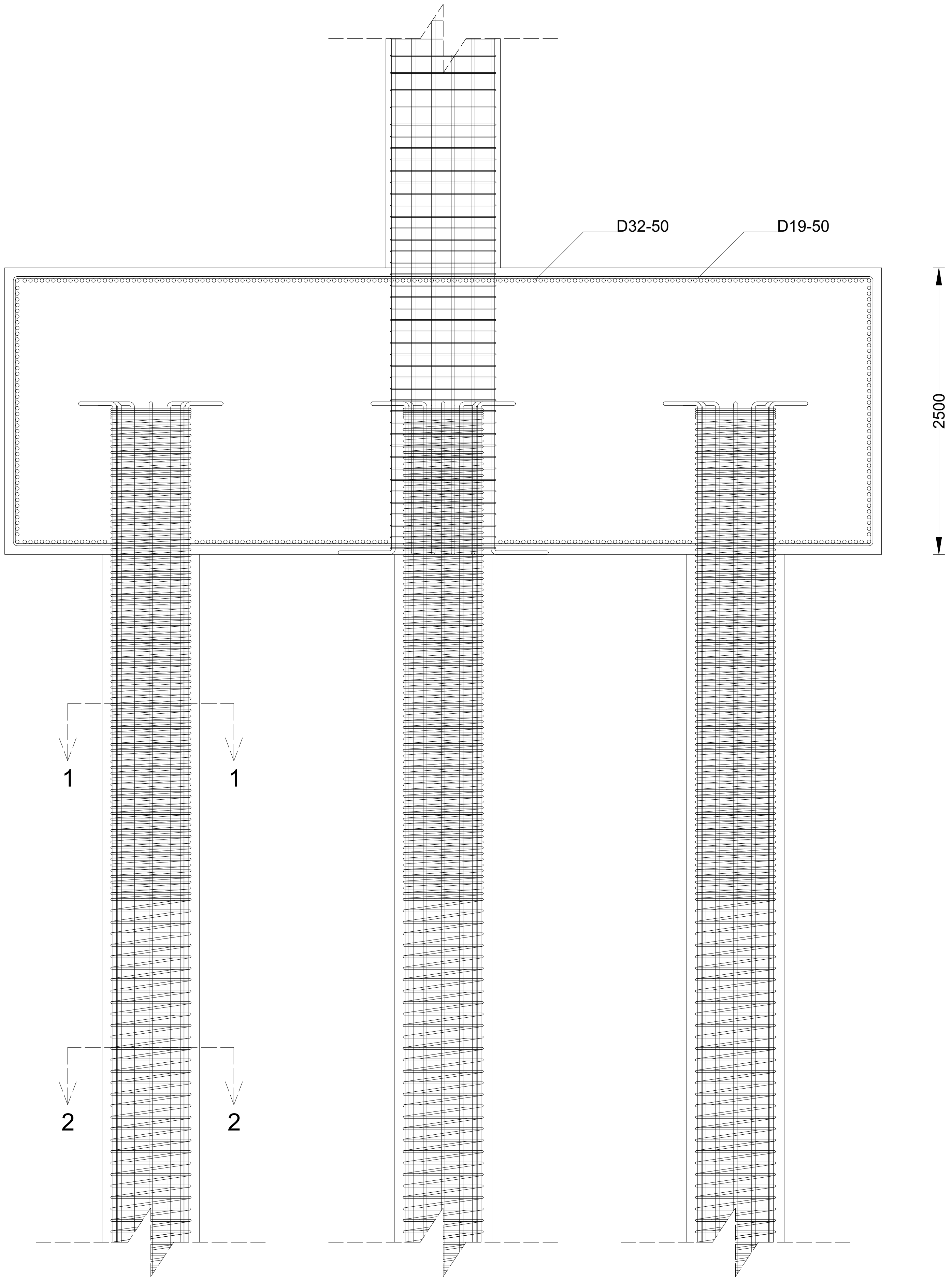

FINAL PROJECT

COMPARATIVE ANALYSIS
BETWEEN INTERNAL FORCES
OF FIXED AND FLEXIBLE
FOUNDATION UNDER
DYNAMIC LOAD

DRAWING TITLE

SECTION CUT A-A
FOUNDATION
REINFORCEMENT DETAIL

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MILLIMETER UNIT

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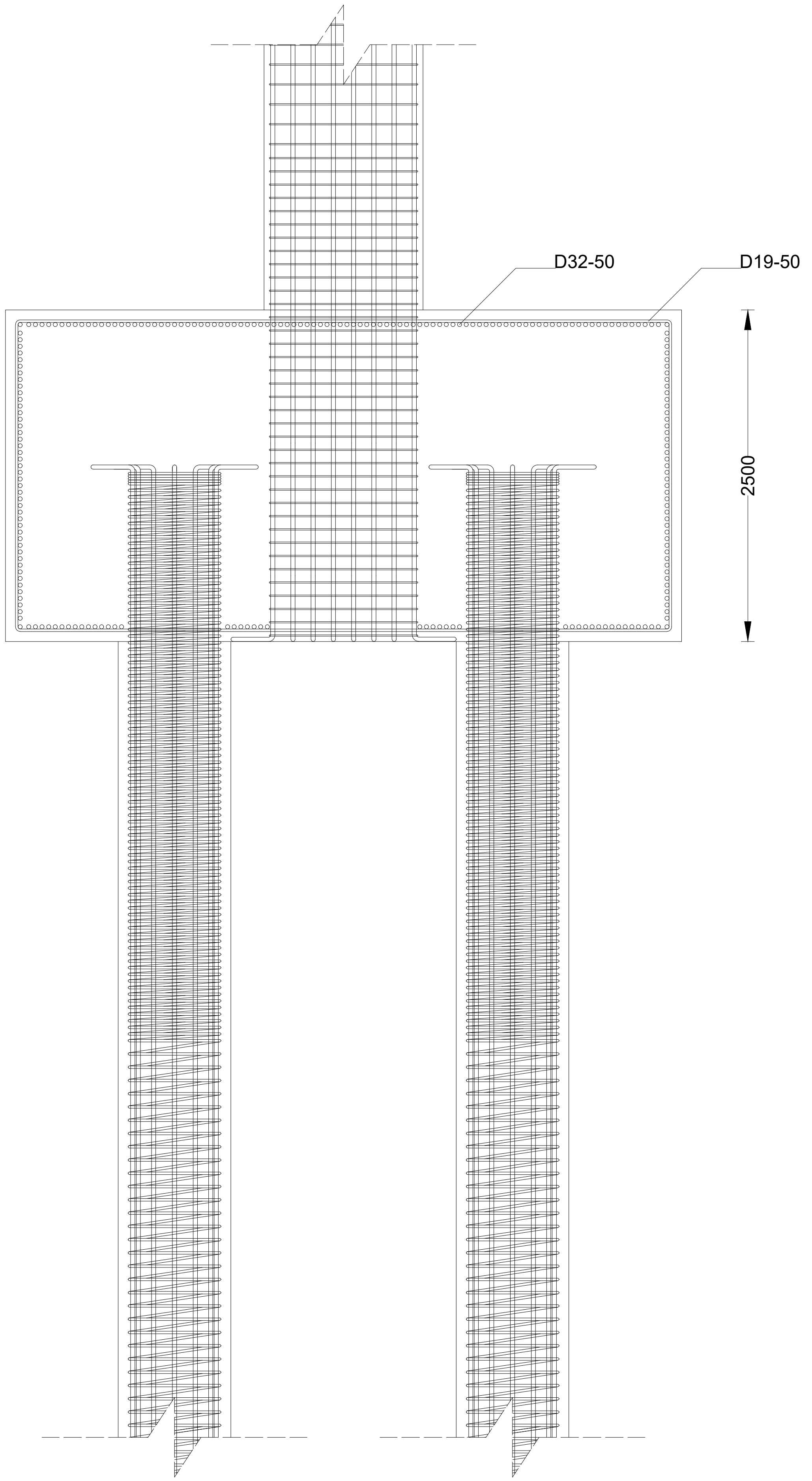

FINAL PROJECT

COMPARATIVE ANALYSIS BETWEEN INTERNAL FORCES OF FIXED AND FLEXIBLE FOUNDATION UNDER DYNAMIC LOAD

DRAWING TITLE

SECTION CUT B-B
FOUNDATION
REINFORCEMENT DETAIL

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PAGE	SCALE
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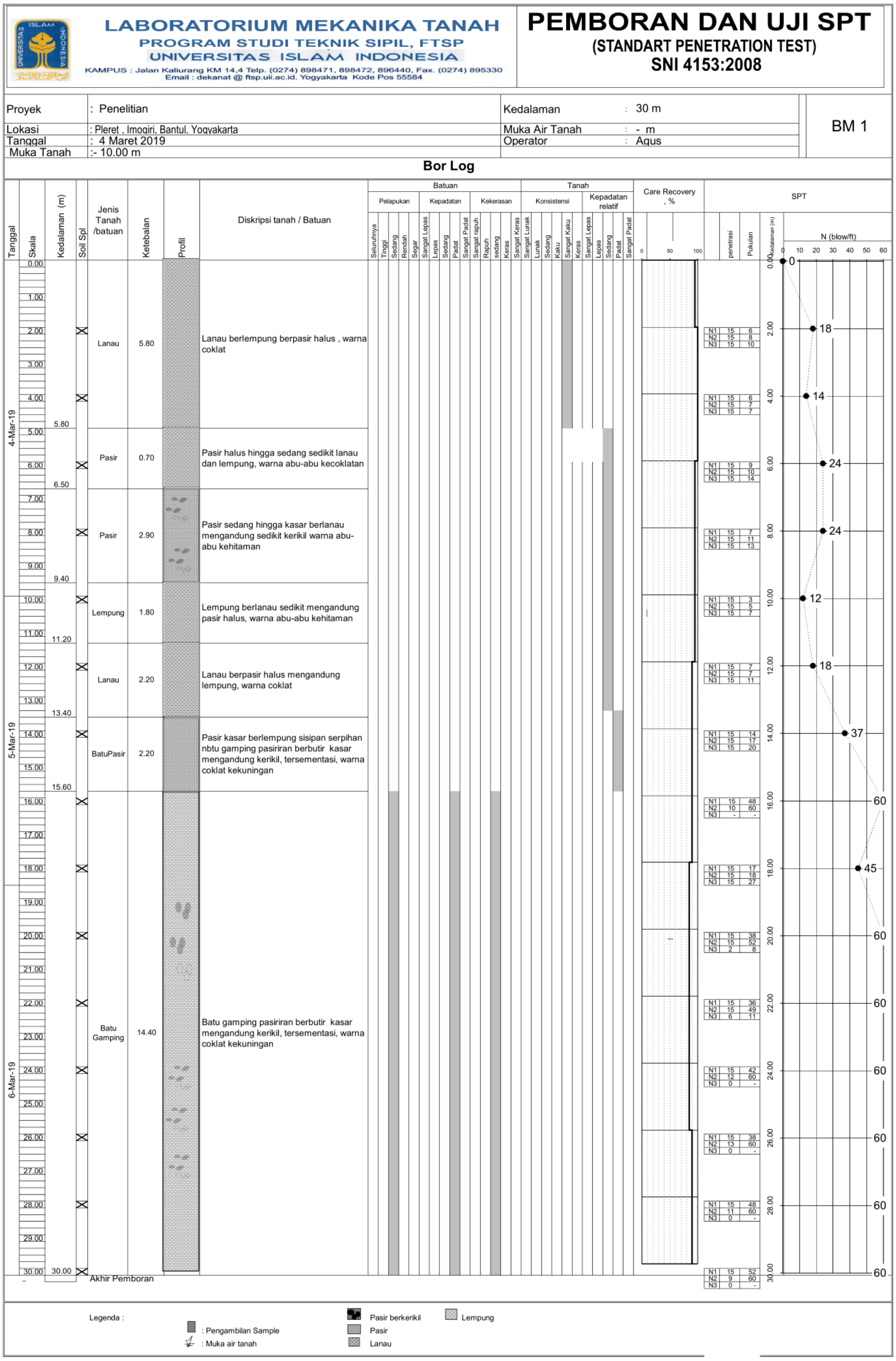
Appendix 19 N-SPT Data



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Email : dekanat @ ftsp.uii.ac.id. Yogyakarta Kode Pos 55584

PEMBORAN DAN UJI SPT
(STANDART PENETRATION TEST)
SNI 4153:2008

Proyek : Penelitian
Lokasi : Pleret , Imogiri, Bantul, Yogyakarta
Tanggal : 4 Maret 2019
Muka Tanah :- 10.00 m
Kedalaman : 30 m
Muka Air Tanah : - m
Operator : Agus

BM 1

Bor Log

Kedalaman (m)	Jenis Tanah /batuan	Ketebalan	Diskripsi tanah / Batuan
5.80	Lanau	5.80	Lanau berlempung berpasir halus , warna coklat
6.50	Pasir	0.70	Pasir halus hingga sedang sedikit lanau dan lempung, warna abu-abu kecoklatan
9.40	Pasir	2.90	Pasir sedang hingga kasar berlanau mengandung sedikit kerikil warna abu-abu kehitaman
11.20	Lempung	1.80	Lempung berlanau sedikit mengandung pasir halus, warna abu-abu kehitaman
13.40	Lanau	2.20	Lanau berpasir halus mengandung lempung, warna coklat
15.60	BatuPasir	2.20	Pasir kasar berlempung sisipan serpihan nbtu gamping pasiriran berbutir kasar mengandung kerikil, tersementasi, warna coklat kekuningan
30.00	Batu Gamping	14.40	Batu gamping pasiriran berbutir kasar mengandung kerikil, tersementasi, warna coklat kekuningan

Akhir Pemboran

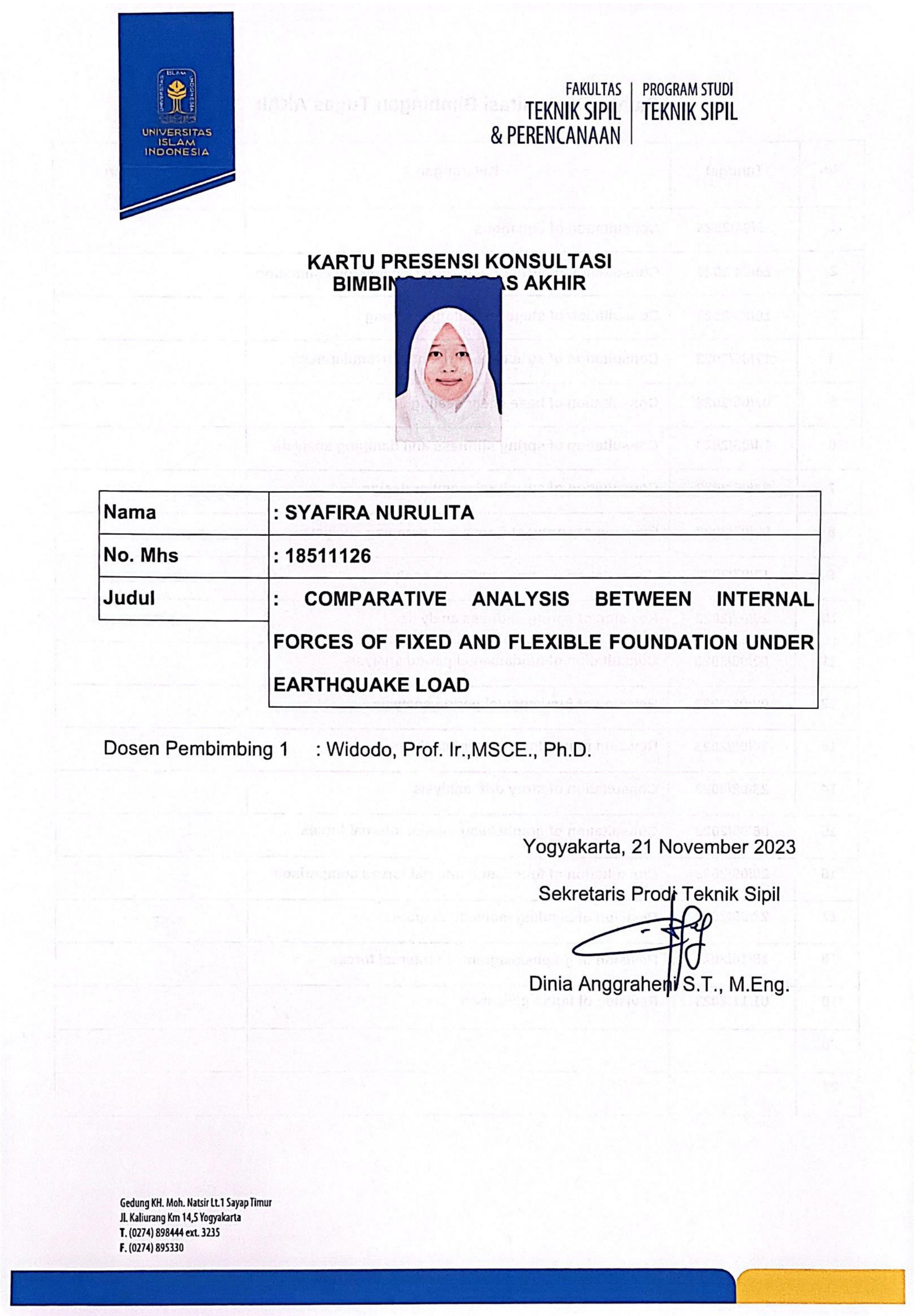
Kedalaman (m)		penetrasi	Pukulan
2.00	N1	15	6
	N2	15	8
	N3	15	10
4.00	N1	15	6
	N2	15	7
	N3	15	7
6.00	N1	15	9
	N2	15	10
	N3	15	14
8.00	N1	15	7
	N2	15	11
	N3	15	13
10.00	N1	15	3
	N2	15	5
	N3	15	7
12.00	N1	15	7
	N2	15	7
	N3	15	11
14.00	N1	15	14
	N2	15	17
	N3	15	20
16.00	N1	15	48
	N2	10	60
	N3	-	-
18.00	N1	15	17
	N2	15	18
	N3	15	27
20.00	N1	15	38
	N2	15	52
	N3	2	8
22.00	N1	15	36
	N2	15	49
	N3	6	11
24.00	N1	15	42
	N2	12	60
	N3	0	-
26.00	N1	15	38
	N2	13	60
	N3	0	-
28.00	N1	15	48
	N2	11	60
	N3	0	-
30.00	N1	15	52
	N2	9	60
	N3	0	-

Legenda :

▮ : Pengambilan Sample
⊻ : Muka air tanah

▩	Pasir berkerikil
▨	Pasir
▦	Lanau
▤	Lempung

Appendix 20 Consultation Card



UNIVERSITAS ISLAM INDONESIA

FAKULTAS TEKNIK SIPIL & PERENCANAAN | PROGRAM STUDI TEKNIK SIPIL

KARTU PRESENSI KONSULTASI
BIMBIN AS AKHIR

Nama	**: SYAFIRA NURULITA**
No. Mhs	**: 18511126**
Judul	**: COMPARATIVE ANALYSIS BETWEEN INTERNAL FORCES OF FIXED AND FLEXIBLE FOUNDATION UNDER EARTHQUAKE LOAD**

Dosen Pembimbing 1 : Widodo, Prof. Ir.,MSCE., Ph.D.

Yogyakarta, 21 November 2023
Sekretaris Prodi Teknik Sipil

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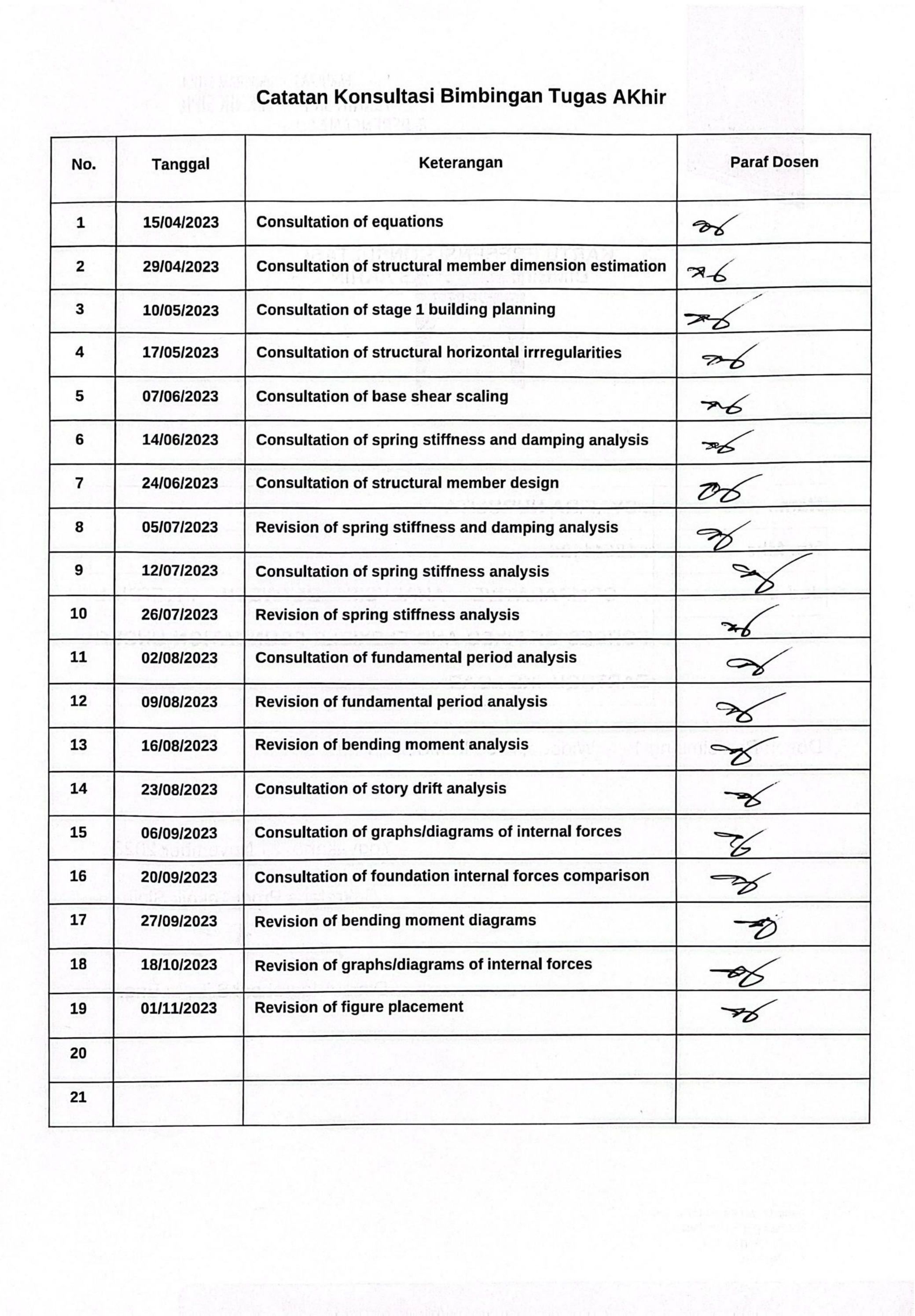
Figure A-1.1 Consultation Card Front Page



Catatan Konsultasi Bimbingan Tugas AKhir

No.	Tanggal	Keterangan	Paraf Dosen
1	15/04/2023	Consultation of equations	
2	29/04/2023	Consultation of structural member dimension estimation	
3	10/05/2023	Consultation of stage 1 building planning	
4	17/05/2023	Consultation of structural horizontal irrregularities	
5	07/06/2023	Consultation of base shear scaling	
6	14/06/2023	Consultation of spring stiffness and damping analysis	
7	24/06/2023	Consultation of structural member design	
8	05/07/2023	Revision of spring stiffness and damping analysis	
9	12/07/2023	Consultation of spring stiffness analysis	
10	26/07/2023	Revision of spring stiffness analysis	
11	02/08/2023	Consultation of fundamental period analysis	
12	09/08/2023	Revision of fundamental period analysis	
13	16/08/2023	Revision of bending moment analysis	
14	23/08/2023	Consultation of story drift analysis	
15	06/09/2023	Consultation of graphs/diagrams of internal forces	
16	20/09/2023	Consultation of foundation internal forces comparison	
17	27/09/2023	Revision of bending moment diagrams	
18	18/10/2023	Revision of graphs/diagrams of internal forces	
19	01/11/2023	Revision of figure placement	
20			
21			

Figure A-1.2 Consultation Card Back Page

Appendix 21 Plagiarism Check Result Statement Letter

Direktorat Perpustakaan Universitas Islam Indonesia Gedung Moh. Hatta
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SURAT KETERANGAN HASIL CEK PLAGIASI

Nomor: 2241773868/Perpus./10/Dir.Perpus/IX/2023

Bismillaahirrahmaanirrahiim

Assalamualaikum Wr. Wb.

Dengan ini, menerangkan Bahwa:
Nama: SyafiraNurulita
NomorMahasiswa :18511126
Pembimbing :Widodo,Prof.Ir.,MSCE.,Ph.D.
Fakultas/Prodi :TeknikSipildanPerencanaan/TeknikSipil(IP)
Judul Karya Ilmiah : COMPARATIVE ANALYSIS BETWEEN INTERNAL FORCES OF FIXED AND FLEXIBLE FOUNDATION UNDER EARTHQUAKE LOAD

Karya ilmiah yang bersangkutan di atas telah melalui proses cek plagiasiTurnitin menggunakan dengan hasil kemiripan (*similarity*) sebesar 5 (Lima) %.

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Wassalamualaikum Wr. Wb.

Yogyakarta,11/29/2023
Direktur

Muhammad Jamil, SIP.