FINAL PROJECT

COMPARATIVE STUDY ON THE INFLUENCE OF SECONDARY BEAM CONFIGURATIONS TO STRUCTURE RESPONSE OF MULTI STORY BUILDING

STUDI KOMPARATIF PENGARUH KONFIGURASI BALOK SEKUNDER TERHADAP RESPON STRUKTUR GEDUNG BERTINGKAT)

Submitted To the Universitas Islam Indonesia Yogyakarta to Fulfil It Requirements For Obtaining A Bachelor's Degree In Civil Engineering

Kautsar Tegar Anugerah
21511004

CIVIL ENGINEERING STUDY PROGRAM
FACULTY OF CIVIL ENGINEERING AND PLANNING
UNIVERSITAS ISLAM INDONESIA
2025

FINAL PROJECT

COMPARATIVE STUDY ON THE INFLUENCE OF SECONDARY BEAM CONFIGURATIONS TO STRUCTURE RESPONSE OF MULTI STORY BUILDING

(STUDI KOMPARATIF PENGARUH KONFIGURASI BALOK SEKUNDER TERHADAP RESPON STRUKTUR GEDUNG BERTINGKAT)

Submitted To the Universitas Islam Indonesia Yogyakarta to Fulfil It Requirements For Obtaining A Bachelor's Degree In Civil Engineering

Kautsar Tegar Anugerah
21511004

CIVIL ENGINEERING STUDY PROGRAM
FACULTY OF CIVIL ENGINEERING AND PLANNING
UNIVERSITAS ISLAM INDONESIA

2025

PLAGIATION FREE STATEMENT

I declare truly that the Final Assignment report is I compiled it as a requirement for completing the bachelor's program in the Study Program Civil Engineering, Faculty of Civil Engineering and Planning, Islamic University of Indonesia is my own work. As for certain parts in writing a Final Assignment report that I quoted from other people's work written in the source clearly in accordance with norms, rules and ethics writing scientific papers. If in the future it is found in whole or in part This final assignment report is not my own work or there are internal plagiarism certain parts, I am willing to accept sanctions, including revocation of title My academic qualifications are in accordance with applicable laws and regulations.

Yogyakarta, August 22,2025

Who make a statement,

METERAI TEMPEL 10000 18A57ANX052656020

Kautsar Tegar Anugerah

(21511004)

FINAL PROJECT

COMPARATIVE STUDY ON THE INFLUENCE OF SECONDARY BEAM CONFIGURATIONS TO STRUCTURE RESPONSE OF MULTI STORY BUILDING
(*STUDI KOMPARATIF PENGARUH KONFIGURASI BALOK SEKUNDER TERHADAP RESPON STRUKTUR GEDUNG BERTINGKAT)*

Arranged by

Kautsar Tegar Anugerah
21511004

Has been accepted as one of the requirements to obtain
a bachelor's degree in civil engineering

Assessed on August 11, 2025
By the Board of Reviewer

Mentor	Reviewer I	Reviewer II
		21.08.25
Jafar, S.T., M.T., MURP.	**Ir. Yunalia Muntafi, S.T., M.T., Ph.D. (Eng)., IPM**	**Astriana Hardawati, S.T., M.Eng.**
NIK: 185111305	**NIK: 095110101**	**NIK: 165111301**

Approved,
Head of Civil Engineering Program

22/8 2025

Ir. Yunalia Muntafi, S.T., M.T., Ph.D. (Eng)., IPM
NIK: 095110101

FOREWORD

Bismillaahirrahmaanirrahiim.

Alhamdulillahirabbil'alamiin, the author gives thanks to Allah for the author was able to complete the final project. The final project is one of the academic requirements for completing degree studies undergraduate degree in Civil Study Program, Faculty of Civil Engineering and Planning, Universitas Indonesian Islam, Yogyakarta.

The final project preparation. There are many obstacles faced by the author, but thanks to suggestions, criticism, and encouragement from various parties, thank to Allah this final project can be completed. In this regard, the author would like to express the deepest thanks to:

1. Mr Fathul Wahid as the rector of Universitas Islam Indonesia,
2. Prof. Ar. Dr.-Ing. Ir. Ilya Fadjar Maharika, MA., IAI. As the Dean of the Faculty of Civil Engineering and Planning of Universitas Islam Indonesia,
3. Ir. Yunalia Muntafi, S.T., M.T., Ph.D. (Eng)., IPM., as the Head of civil engineering undergraduate Study program and academic supervisor that gave practical work advice, academic advice, academic recommendations and as Reviewer I of this final project,
4. Pradipta Nandhi Wardana, S.T., M.Eng., as the Secretary of civil engineering international undergraduate program that gave academic principal advice,
5. Jafar, S.T., M.T., MURP., as the Mentor of the final project that gave final project advice and recommendation,
6. Astriana Hardawati, S.T., M.Eng., as the Reviewer II of this final project,
7. All Universitas Islam Indonesia lecturer that gave the knowledge and experience to author,
8. All Universitas Islam Indonesia secretary, academic, and other employee that gave study service, academic service, and other University services.
9. Agung Mulyanto as the owner of Nata Hotel who gives the hotel building plan data for the research,

10. Mohammad Sadli, S.T., M.T., that helped author, both materially and spiritually, until the completion of this Final Project, and
11. The author's family that has helped author, spiritually, until the completion of this Final Project.

Finally, the author hopes that this final project can be useful for various parties who read it.

Yogyakarta, August 22, 2025

Author,

Kautsar Tegar Anugerah

(21511004)

TABLE OF CONTENT

LIST OF IMAGES

LIST OF TABLES

LIST OF NOTATION AND ABBREVIATIONS

Δ	=	Drift (δ_i -δ_{i+1}) (m)
A_g	=	Gross area (mm^2)
C_d	=	Deflection amplification factor
CoM	=	Centre of Mass
C_s	=	Seismic Coefficient Response
C_{vx}	=	Vertical distribution factor
d_i	=	Seismic force resisting system horizontal dimension (m)
E_c	=	Elastic modulus concrete (MPa)
E_y	=	Elastic modulus steel (MPa)
F_a	=	Site coefficient for short period of 0.2 seconds
f_c'	=	Concrete Strength (MPa)
F_v	=	Site coefficient for long period of 1 second
F_x	=	Lateral Seismic Force (kN)
f_y	=	Steel Strength (MPa)
γ_{rc}	=	Reinforced concrete volume weight (kN/m^3)
h_b	=	Beam thickness (mm)
h_s	=	Slab thickness (mm)
h_{sx}	=	Height of story below story x (m)
i_b	=	Inertia of beam (mm^4)
I_e	=	Importance Building Factor
i_s	=	Inertia of slab (mm^4)
k	=	Exponent of structure period
K	=	Stiffness (kN/m) or (kN/mm)
l	=	Length (m)
l_x	=	Slab shorter span length (mm)
l_y	=	Slab longer span length (mm)
MCE_R	=	Maximum Considered Earthquake Targeted Risk
P	=	Vertical design load (kN)

R = Response Modification Factor

S_1 = MCE_R earthquake spectral acceleration response parameters are mapped for 1 second period (g)

S_a = Acceleration spectral response caused by soil-structure interaction effects (g)

S_{D1} = One period of design spectral response acceleration parameter (g)

S_{DS} = Short periods of design spectral response acceleration parameter (g)

S_{M1} = MCE_R. 5%-damped, period of 1 s adjusted spectral response acceleration parameter for site effects

S_{MS} = MCE_R. 5%-damped, short period spectral response acceleration parameter for site effects

S_s = MCE_R earthquake spectral acceleration response parameters are mapped for short period (g)

T = Structure fundamental period (s)

T_0 = Lower boundary of period(s)

T_a = Approximate Structure Period(s)

T_c = Structural analysis program result of structure fundamental period(s)

TIR = Torsional Irregularity Ratio

T_L = Long-period transition period(s)

T_s = Upper boundary of period(s)

V = Base Shear (kN)

V = Structure base shear (kN)

W = Structure Weight (kN)

x = The reviewed story level

β = Ratio of shear requirement to shear capacity for stories between stories

δ_x = Deflection inelastic (m)

δ_{xe} = Deflection elastic (m)

θ = Stability Coefficient

Ω = Structure system overstrength factor

ABSTRACT

Secondary beam is the non-primary structural elements in building that commonly applied to structures that have long span frame. The principle of the secondary beam is to transfer the structure load to the primary beam. Since the non-primary structural elements, the secondary beam is designed with the purpose of structural efficiency rather than structural earthquake-resistant capacity. Therefore, a comparison of the influence of several secondary beam configurations on the structural response of the building is observed with principles of earthquake-resistant building in this study. The results are expected to provide information regarding the influence of secondary beams on the 7-story building that applies earthquake-resistant structure principles.

Structural members are designed with minimum height by considering structural member span (SNI 2847-2019). Secondary beam configurations 1 to 5 are secondary beam configurations, without secondary beam, on weak axis, strong axis, both weak and strong axes, and both diagonal axes, respectively. Structure model variations with 5 secondary beam configurations are evaluated by earthquake resistant building code (SNI 1726-2019). Structure response result of structure with 5 secondary beam configurations are compared to conclude secondary beam influence in structure.

The result of research shows that the configuration of secondary beam in the structure influences the structure response. Structure with secondary beam tend to have lower structure volume and more period (T) except secondary beam configuration 5. The structural response resulted in deflection and P-delta increases for model variation 2 to 4 and deflection and P-delta decreases for model variation 5 compared to model variation 1 as a control.

Keyword: Secondary beam, Structure response, Story drift, P-delta effect, Irregularities

ABSTRAK

Balok sekunder merupakan elemen struktural non-primer pada bangunan yang umumnya diterapkan pada struktur dengan rangka bentang panjang. Prinsip kerja balok sekunder adalah menyalurkan beban struktur ke balok primer. Balok sekunder merupakan elemen struktural non-primer, balok sekunder dirancang dengan tujuan efisiensi structural daripada kapasitas struktural tahan gempa. Oleh karena itu, dalam studi ini, perbandingan pengaruh beberapa konfigurasi balok sekunder terhadap respons struktural bangunan diamati dengan prinsip-prinsip bangunan tahan gempa. Hasil penelitian ini diharapkan dapat memberikan informasi mengenai pengaruh balok sekunder terhadap bangunan 7 lantai yang menerapkan prinisip struktur tahan gempa.

Bagian-bagian strukur didesain dengan ketebalan minimum dengan mempertimbangkan bentang anggota struktur (SNI 2847-2019). Konfigurasi balok sekunder 1 sampai 5 berturut-turut adalah konfigurasi balok sekunder, tanpa balok sekunder, pada sumbu lemah, pada sumbu kuat, pada kedua sumbu lemah dan kuat, serta diagonal. Variasi model struktur dengan 5 konfigurasi balok sekunder dievaluasi dengan pertaturan bangunan tahan gempa (SNI 1726-2019). Hasil respon struktur dengan 5 konfigurasi balok sekunder dibandingkan untuk menyimpulkan pengaruh balok sekunder pada struktur.

Hasil penelitian menunjukkan bahwa konfigurasi balok sekunder pada struktur mempengaruhi respon struktur. Struktur dengan balok sekunder cenderung memiliki volume struktur yang lebih sedikit dan periode (T) yang lebih besar kecuali pada konfigurasi balok sekunder 5. Respon strukutr yang dihasilkan berupa peningkatan defleksi dan P-delta untuk variasi model 2 sampai 4 dan penurunan lendutan pada P-delta untuk variasi model 5 dibandingkan dengan variasi model 1 sebagai kontrol.

Kata kunci: *Balok sekunder, Respon struktur, Simpangan lantai, Efek P-delta, Iregularitas*

CHAPTER I
INTRODUCTION

1.1 Background

Indonesia geologically located at the meeting point of three major tectonic plate and Pacific Ring of Fire path. Therefore, Indonesia is vulnerable to earthquake, volcanoes, and tsunami. The geological phenomena that occur in the present, namely the increase of faults and earthquake zones (Rakuasa & Pakniany, 2024). One of the phenomena that people are concerned recently is a large subduction fault known as a Megathrust. Megathrusts can cause tsunamis and large earthquakes (Widiyantoro et al., 2020). Mitigating and reducing the impact of disasters is an effort that must be carried out by humans. Consequently, planning the building in Indonesia must satisfy the standard about the Earthquake-resistant building.

The concrete frame is the most common used building structure in the field. The reason of it is because the concrete frame is easy to design, easy to implement, and more economical to build in the field (Wight, 2016). The structure of the building consists of sub structure and upper structure. The sub structure is the structure component located underground. The sub structure consists of foundations, tie beam, retaining wall, etc. Upper structure is the structure component located above the ground. The Upper structure consists of column, beam, floor plate, shear wall, etc.

The multi-story building needs the structure ductility and stiffness to withstand the earthquake load. The ductility will cause the building not brittle. The earthquake load base shear transmits to upper floor of the building and become the story horizontal force (Pawirodikromo, 2012). Floor mass of each story is the one of the horizontal load parameters.

Planning a building requires consideration of design and materials to achieve a strong and safe building for occupant. Therefore, planning the structure must consider the aspect of structural, budget, esthetical, and function of its structure. The method of cost saving in the project is implementation of secondary beam. The secondary beam used for the structure that have long span portal. Theoretically, the longer span of portal will cause larger floor plate flexural moment. Hence, increasing the thickness of floor plate is the solution to reduce the floor plate flexural moment. However, increasing the floor plate thickness will increase the dead load of the structure and will affect overall dimensional specification of structure. The application of secondary beam will distribute the load from the plate to main beam. So, the thickness of the floor plate can be reduced because the span of the floor plate is reduced also.

The configuration of secondary beam should be considered to achieve optimal structure design. The writer will do research about configuration of secondary beam in multi-story building. The research will aim the influence of secondary beams configuration on the structural response, i.e. the structure deflection, structure drift, and the P-delta effect. That structural response will be compared by considering the structure volume.

1.2 Problem Formulation

The problem formulation relies on the structural design. The problem formulation has a role to limit the research. The problem formulations are as follows.

1. How the structure with secondary beam configuration variations affects the structure response?
2. How the structure with secondary beam configuration variations affects the structure irregularities?
3. How the structure with secondary beam configuration variations affects the structure story drifts?
4. How the structure with secondary beam configuration variations affects the P-delta effect?

5. What is the influence of the secondary beam configuration variations considering by the structure response and structure volume?

1.3 Objectives

The chosen objective is a description of the purpose of the problem formulation. The Objectives of the research are as follow.

1. Know the comparison of structural responses between the structure with secondary beam and without secondary beam.
2. Know the comparison of structural irregularities between the structure with secondary beam and without secondary beam.
3. Know the effect of the application of secondary beam configuration variations in structure story drifts.
4. Know the effect of the application of secondary beam configuration variations in P-delta effects.
5. Know the influence of secondary beam configuration variations by considering the structure response and structure volume ratio.

1.4 Research Benefit

This research is expected to help give information to planners, foremen, and workers about consequences in implementation of secondary beam in multi storey building by considering the structure response data of multi-storey building with secondary beams application. Contribute to test the theory of earthquake-resistant building by achieve the information and data regarding the influence of the secondary beam in the structure response.

1.5 Scope of Research

The content has the scope of the research, to obtain the specific result of the research. The limitations of the research are following.

1. The model plan is taken from existing building.
2. The existing building data used is only the structural plan.
3. The dimensions of the beam, plate, and column is determined based on minimum dimension requirement in accordance with the code (BSN, 2019b).

4. The taken model will modify with the configuration of secondary beams.
5. Building risk category based on the building purpose in accordance with the code (BSN, 2019a).
6. Building location in East Java.
7. The soil type is SD in accordance with USGS *v30s* mosaic value at Ngawi, East Java, Indonesia is around 200 m/s.
8. The load analysis considers dead load (w_D), live load (w_L), and earthquake load (w_E).
9. Frame support set as fix support, so the foundation rotation is ignored.
10. Concrete quality is set to f_c' = 25 MPa and steel quality uses f_y = 420 MPa.
11. SAP2000 or Autodesk Robot is used to analyse the structure.
12. Secondary beam configuration will have 5 variations; the configuration variations are as follows.
    a. Configuration without secondary beam on building layout.
    b. One secondary beam in weak axis of building layout.
    c. One secondary beam in strong axis of building layout.
    d. Secondary beam in both axes of building layout.
    e. Secondary beams in both diagonal axes of building layout.

CHAPTER II
LITERATURE REVIEW

2.1 General Description

Literature Review is a part of scientific work that contains a systematic description of previous research or scientific work. The literature review will relate to the research or scientific work that will be carried out in this scientific work. Sources in the literature review will be written by stating the author's name and year of publication.

2.2 Previous Research

Previous research will be reviewed as a reference in writing this scientific work. The references to be reviewed are scientific papers in the form of research journals, theses, and final assignments. Previous research reviewed for reference in writing this scientific work is as follows.

Budiyawan & Yulandaru (2006) attempted to determine the effect of the placement of secondary beams and floor plate thickness on main beams. The building model has the variation of secondary beam configuration with reinforced concrete structure. The discussion of this research analyses the area of reinforcement designed from the effect of the secondary beams on the forces occurring in the main beam. The result of the research is that the more secondary beams used, the less reinforcement is required by the secondary beams and the reinforcement of the floor slab.

Shuraim (2003) attempted to apply the design code method for reinforced concrete slabs to secondary beams. The loading in this study only applied a uniform load of 15 kN/m^2. There are only two types of secondary beam configurations reviewed, namely structures without secondary beams and structures with secondary beams. This research is more focused on examining the effect of the thickness of the secondary beam using the Beam to Slab depth Ratio (BSR) approach to the variation of the moment obtained from the SAP2000 software analysis. The discussion in this research shows that the greater the beam to slab depth ratio, the greater the percentage of moment received by the secondary beam. The research results can be seen in Figure 2.1.

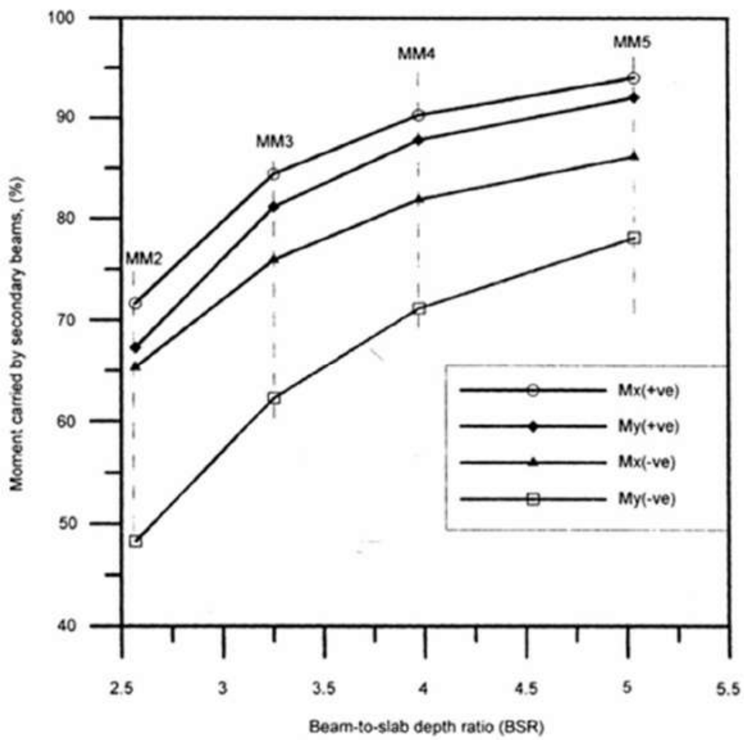

Figure 2.1 Moment Carried by Secondary Beam (%) to BSR

Source: (Shuraim, 2003)

Based on Figure 2.1 the results of the study can be concluded that the more depth of the secondary beam, the greater the contribution of the secondary beam.

Yanita & Fajarwati (2021). attempted to examines the addition of secondary beams to the volume of floor slabs. The purpose of this research was to find the most effective addition of secondary beams to the volume of material. The discussion in this study is the addition of secondary beams and the reduction of floor slab thickness considering the deflection of the slabs. The result obtained from this research is that the addition of secondary beams can reduce the volume of material if the span of the floor slab is wide.

Baek (2016) attempted to examines the response of reinforced concrete to the structural components it supports and the indirect loads from secondary beams. This study aims to determine the reinforcement requirements of hanging beams due to indirect loading and to see how they match the CSA Standard A23.3-14 design equation. This research tested the specimens by means of compressive strength testing on the specimens. The testing setup in this study can be seen in the following Figure 2.2.

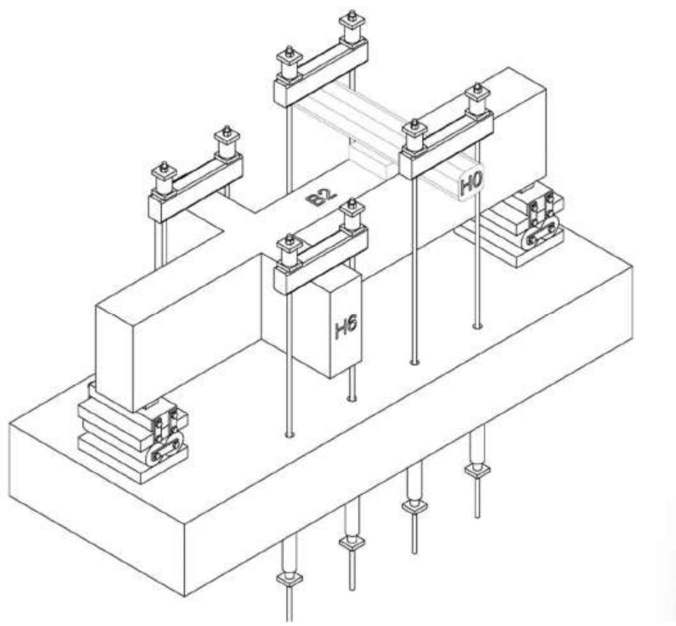

Figure 2.2 Indirect Load Test Setup

Source: (Baek, 2016)

The discussion of this research results in that shear reinforcement is placed in the beam connection as well as placed adjacent to the beam connection in accordance with CSA Standard A23.3-14. The results obtained from the research are that the tensile force from the calculation of CSA Standard A23.3-14 is higher than the test results. So, the CSA Standard A23.3-14 calculations are safe to apply.

Uzodimma (2016) attempted to examines the design and influence of a network interacting primary and secondary beams as an alternative in the hall of 12m x 20m dimension. The model's load transfer from secondary beams to primary beam, structural analysis, and the reinforcement design was analysed manually. The result obtained from this research is that the secondary beam and primary beam network recommend as an alternative in large span construction, rather than add the thickness of the slab. The research provides adequate design and detailing of the element member.

2.3 Research Differences

Based on the review of previous research, there are differences with the research that will be carried out in this scientific work. This scientific work will analyse the effect of secondary beams on the structure response of building including irregularities, story drift, P-delta effect and the load applied is load combination including dead loads, live loads, and earthquake loads. While the previous research examines the local element member response, i.e., slab and main beam. The differences between previous research and this research are plotted in Table 2.1.

Table 2.1 Literature Review Matrix

Description	Authors				
	Ahmed B. Shuraim, (2003)	M. Budiyawan, Purnawan Yulandru, (2006)	Kyoung Rae Baek, (2016)	Uzodimma (2016)	Rachmi Yanita, Sekar Fajarwati, (2021)
Research Title	Applicability of Code Design Methods to RC Slabs on Secondary Beams. Part II: Moment Variations	*Pengaruh Variasi Peletakan Balok Anak Terhadap Volume Struktur Pelat dan Balok*	Response of Reinforced Concrete Beams with Indirect Loading.	Analysis and Design of a Network of Interacting Primary and Secondary Beams as Alternatives in Large Span Construction	*Efektifitas Penambahan Balok Anak Terhadap Volume Material Konstruksi Pelat Lantai Beton Bertulang*
Research Objectives	Apply the design code method for reinforced concrete slabs to secondary beams. examine the effect of the thickness of the secondary beam using the Beam to Slab depth Ratio (BSR) approach to the variation of the moment	Know the level of efficiency or economy between the addition of plate thickness and the application of secondary beam in a building. Know the influence of secondary beam placement variations on the main beam in a building.	Achieve the response of hanging reinforced concrete beams due to indirect loading	Examine the design and influence of secondary beam for structure with long span slab	Achieve the most effective floor slab volume material when the additional secondary beams applied
Research Method	- Apply 15 kN/m^2 as the structure uniform load - Review the internal forces of the element	- Research location. - Earthquake zone category III. - Stiff soil. - Analysis using SAP2000.	- The beams are tested in laboratory with applying the indirect model load - The analysis and testing in accordance with the CSA code	- Application of secondary beam for 12m x 20m hall structure. - Analysed manually the load transfer from the slab and secondary beam to the primary beam - Analysed the reinforcement design.	- Floor plate is reinforced concrete with variation - Analyse the force use structural analysis program - Compare the plate deflection and material volume

Table 2.1 (Continuing) Literature Review Matrix

Research Result and Conclusion	- Beam to Slab depth ratio (BSR) - Moment received by primary beam from secondary beam	- Longitudinal reinforcement of beam design.	- Tensile force from the calculation of CSA standard is higher than the test results - CSA Standard calculation is safe to apply in the indirect load condition	- The structure with secondary beam is more efficient in the reinforcement design rather than without secondary beam - Application of secondary beam is more efficient rather than adding the thickness of the slab	- Secondary beam application reduces the material volume of plate construction - Secondary beam application reduces the deflection
Difference	- The load includes earthquake behaviour. - The final product is the comparison of secondary beam configuration on structural response, irregularities, P-delta effect.	- The final product is the comparison of secondary beam configuration on structural response, irregularities, P-delta effect.	- The objects is a whole structure by evaluate the structure response - The final product is the comparison of secondary beam configuration on structural response, irregularities, P-delta effect.	- The application of secondary beams is in several configuration. - The final product is the comparison of secondary beam configuration on structural response, irregularities, P-delta effect	- The parameters used is the structural volume and structure response - The final product is the comparison of secondary beam configuration on structural response, irregularities, P-delta effect. -

2.4 Hypothesis

According to the literature review and theoretical basis the hypothesis of the research is as follows.

1. The structure without secondary beams is the structure variation with the most ductility resulting structure with the most drift.
2. The structure with secondary beams has thinner slab resulting in fewer structure mass and structure response decrease.
3. The structure with secondary beams at strong axis will have less influence in the structural response.
4. The structure with secondary beams at weak axis will have more influence in structural response.
5. The structure with secondary beams at both axes will have the most influence in structural response.

CHAPTER III
THEORETICAL BASIS

3.1 Introduction

The building is designed to achieve the ductility. Structure ductility is the ability of concrete element to deform inelastically due to cyclic load without any significant strength loss (Pawirodikromo, 2012)

3.2 Seismic ground motion

Seismic ground motion is the movement of the earth's surface due to earthquake seismic force. The planning refers to SNI 1726-2019 (BSN, 2019a) and ASCE/SEI 7-22 (ASCE, 2021).

3.2.1 Site class

The site class is classified as site class SA, SB, SC, SD, SE, or SF. the classification is referring to the site class classification. The site classification in accordance with

Table 3.1 Site Class Classification

Site Class	**Average v_s (m/second)**	**Average N or N_{ch}**	**Average S_u (kPa)**
SA (hard rock)	>1500	N/A	N/A
SB (rock)	750 – 1500	N/A	N/A
SC (hard soil, very dense, and soft rock)	350 – 750	> 50	≥ 100
SD (medium soil)	175 – 350	15 – 50	50 – 100
SE (soft soil)	< 175	< 15	< 50
	Or any soil profile containing more than 3 m of soil with characteristics as follows: 1. Plastic index, $PI > 20$, 2. Water content, $w \geq 40\%$, and 3. Non-flowing shear strength, $S_u < 25$ kPa		
SF (special soil, require specific soil investigation and specific site response analysis)	Any soil layer profile that has one or more of the following characteristics: - Vulnerable and potentially fail or collapse due to earthquake loads such as easy liquefaction, very sensitive clay, weak cemented soil - Very organic clay and/or peat (thickness $H > 3$ m) - Very high plasticity clay (thickness H > 7.5 m with plasticity index $PI > 75$) Soft/semi-firm clay layer with a thickness of H > 35 m with average $S_u < 50$ kPa		

Source: (BSN, 2019a)

3.2.2 Site coefficient and acceleration spectral response parameter at maximum considered earthquake ground motion (MCE_R)

The determination of earthquake acceleration response MCE_R in the ground level, the seismic amplification factor at 0.2 seconds and 1 second are required (BSN, 2019a). The determination in accordance with equation to (3.1).and (3.2).

$$S_{MS}=F_a S_s \tag{3.1}$$

$$S_{M1}=F_v S_1 \tag{3.2}$$

Where:

S_s = MCE_R earthquake spectral acceleration response parameters are mapped for short period, and

S_1 = MCE_R earthquake spectral acceleration response parameters are mapped for 1 second period.

The value of the S_1 can be obtained in accordance with the ground motion parameters maps for 1 second period as served in the Figure 3.1.

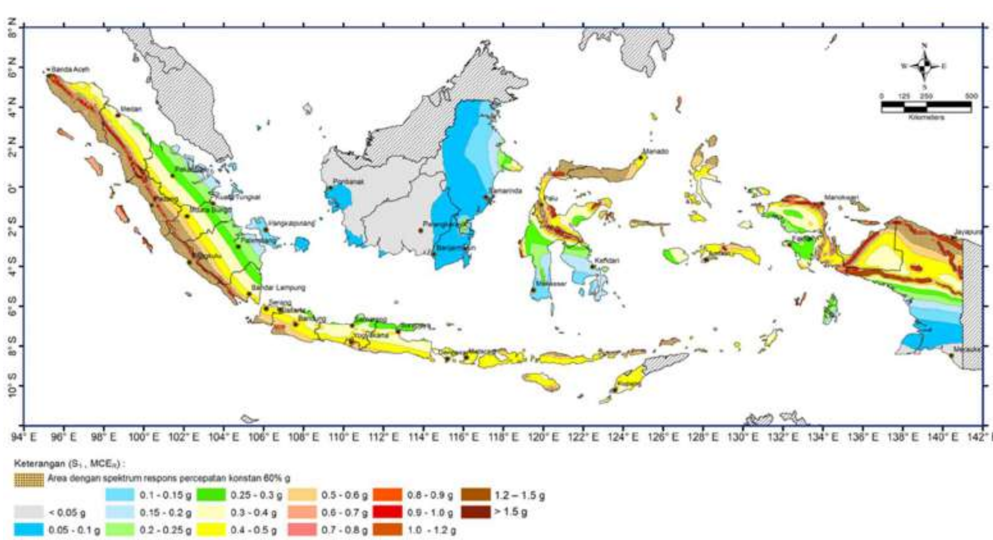

Figure 3.1 Ground Motion Parameter, *S1*, Maximum Considered Targeted-Risk Earthquake (MCE_R) For the Indonesian Region For 1- Second Response Spectrum (Critical Attenuation 5 %)

Source: (BSN, 2019a)

The value of the S_s can be obtained in accordance with the ground motion parameters maps for 0.2 seconds period as served in the Figure 3.2

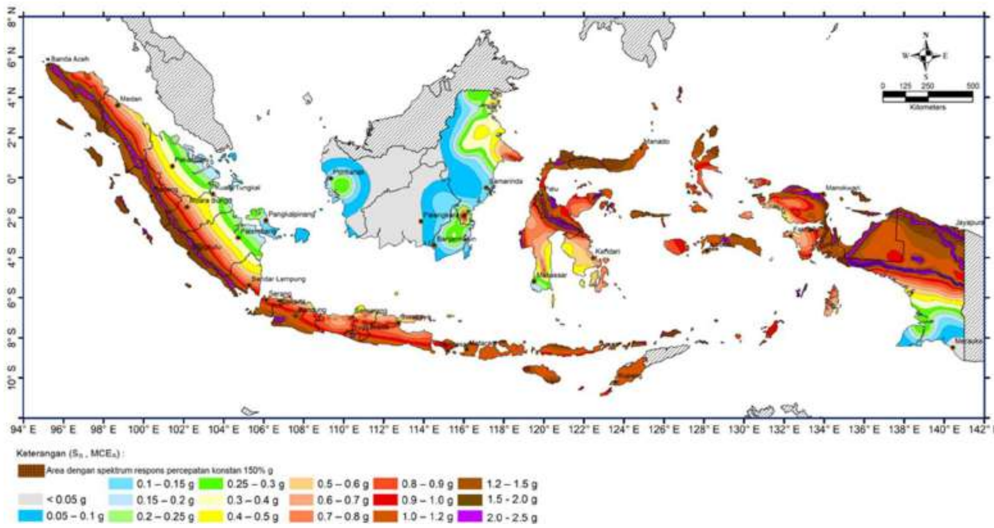

Figure 3.2 Ground Motion Parameter Ss, Maximum Earthquake Considered Targeted-Risk (MCE_R) For the Indonesian Region For 0.2-Second Response Spectrum (Critical Attenuation 5 %)

Source: (BSN, 2019a)

The site coefficient F_a and F_v are served in the following tables as follows. If the site class is SE, the F_a value cannot be less than 1.2 (BSN, 2019a). the F_a and F_v values can be seen in Table 3.2 and Table 3.3.

Table 3.2 Site Coefficient, *Fa*

Site Class	The maximum earthquake acceleration spectral response parameters considered at the targeted risk (MCER) mapped at a short period, T = 0.2 seconds, S_s					
	$S_s \leq 0.25$	$S_s = 0.5$	$S_s = 0.75$	$S_s = 1.0$	$S_s = 1.25$	$S_s \geq 1.5$
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	Sites requiring specific geotechnical investigations and site-specific response analysis					

Source: (BSN, 2019a)

Table 3.3 Site Coefficient, *Fv*

Site Class	Maximum earthquake acceleration spectral response parameters considered at target-risk (MCER) mapped at 1-second period, S_1					
	$S_1 \leq 0.25$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \geq 0.6$
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	Sites requiring specific geotechnical investigations and site-specific response analysis					

Source: (BSN, 2019a)

3.2.3 Spectral acceleration design parameters

The design spectral acceleration parameters are the design at short period of earthquake spectral response acceleration. The equation as follows.

$$S_{DS} = \frac{2}{3} S_{MS} \tag{3.3}$$

$$S_{D1} = \frac{2}{3} S_{M1} \tag{3.4}$$

where:

S_{MS} = MCE_R. 5%-damped, short period spectral response acceleration parameter for site effects, and

S_{M1} = MCE_R. 5%-damped, period of 1 s adjusted spectral response acceleration parameter for site effects.

3.2.4 Two-period design response spectrum

For periods less than T_0, the parameter of acceleration response, S_a, will be taken in accordance with equation as follows.

$$S_a = S_{DS}\left(0{,}4 + 0{,}6\frac{T}{T_0}\right) \tag{3.5}$$

For periods that greater than or equal to T_0 and less than or equal to T_S, the design parameter of acceleration response, S_a, will be taken as equal to S_{DS}.

For periods that greater than T_S and less than or equal to T_L, the design parameter of spectral acceleration response, S_a, will be taken in accordance with equation as follows.

$$S_a = \frac{S_{D1}}{T} \tag{3.6}$$

For periods greater than T_L, S_a will be taken in accordance with equation as follows.

$$S_a = \frac{S_{DS}\,T_L}{T^2} \tag{3.7}$$

Where:

S_{DS} = Short periods of design spectral response acceleration parameter,

S_{D1} = one period of design spectral response acceleration parameter,

T = structure fundamental period,

T_S = $\frac{S_{D1}}{S_{DS}}$,

T_0 = $0{,}2 \times \left(\frac{S_{D1}}{S_{DS}}\right)$, and

T_L = Long-period transition period(s) in accordance with.

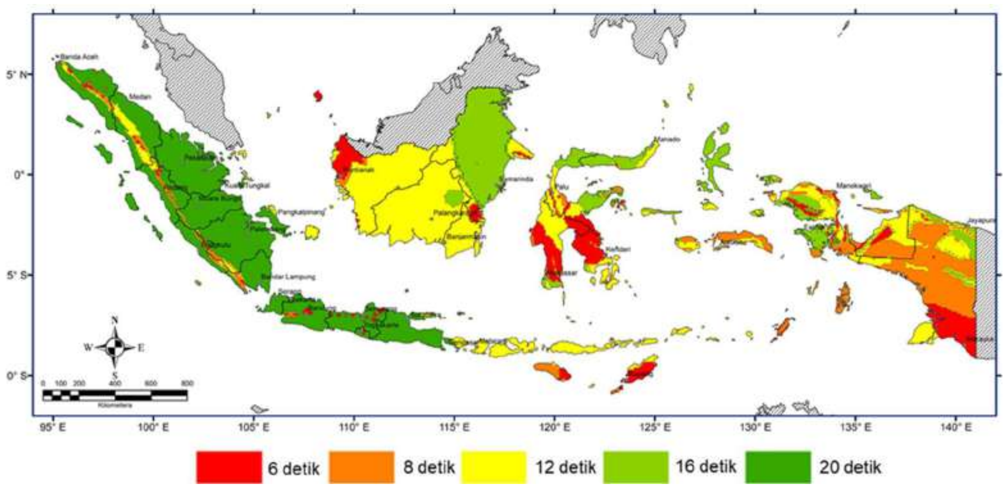

Figure 3.3 Long Period Transition Map, Indonesia Region

Source: (BSN, 2019a)

3.2.5 Response spectrum

Response spectrum is the curve that show the maximum response versus structural frequency relationship (Kumar, 1990). The vertical axis of the spectrum plot is the computation of acceleration in terms of the acceleration due to the gravity. The ordinate axis is the structure natural vibration period (T). The response spectrum plot design can be seen in Figure 3.4.

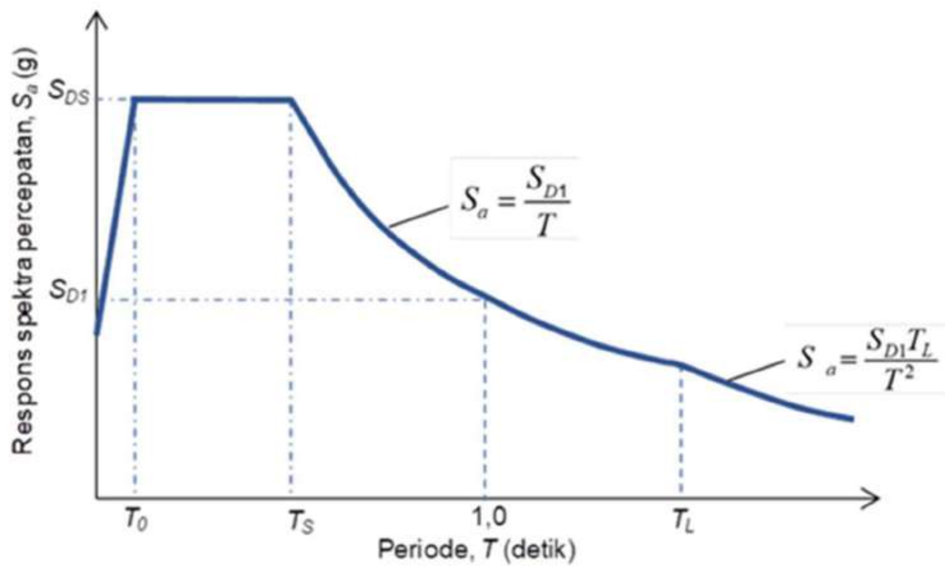

Figure 3.4 Response Spectrum Design

Source:(BSN, 2019a)

3.3 Model Definition

3.3.1 Seismic Design Category

Structure with risk category I, II, or III that located in the mapped acceleration spectral response parameter at a period of 1 second, S_1 is greater than or equal to 0.75 shall be designated as structures of seismic design category E. Structures of risk category IV located in the mapped acceleration spectral response parameter at a period of 1 second, S_1, is greater than or equal to 0.75, shall be designated as structures with a seismic design category of F. While for all other structures shall be assigned a seismic design category based on their risk category and their design acceleration spectral response parameters, S_{DS} and S_{D1}, in accordance with section 3.2.3 (BSN, 2019a). The seismic design category determination with S_{DS} can be seen in Table 3.4. While for the seismic design category determination with S_{D1} can be seen in Table 3.5.

Table 3.4 Seismic Design Category Based on Short-Period Acceleration Response Parameters

Value S_{DS}	Risk Category	
	I or II or 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: (BSN, 2019a)

Table 3.5 Seismic Design Category Based on Acceleration Response Parameters at A Period of 1 Second

Value S_{D1}	Risk Category	
	I or II or III	IV
$S_{D1} < 0.067$	A	A
$0.067 \leq S_{D1} < 0.133$	B	C
$0.133 \leq S_{D1} < 0.20$	C	D
$0.20 \leq S_{D1}$	D	D

Source: (BSN, 2019a)

3.3.2 Structure system

The response modification response, system overstrength factor, deflection amplification factor, and structure system limitation of special moment resisting reinforced concrete frame structure system can be seen in Table 3.6.

Table 3.6 Factor R, Ω, and C_d for Seismic Force Resisting System

Seismic Force Resisting System	Response Modification Response	System Overstrength Factor	Deflection Amplification Factor	Structure System limitation and Structural height limitation, h_n (m)				
				Seismic Design Category				
Moment Resisting Frame System	R	Ω	C_d	**B**	**C**	**D**	**E**	**F**
Special moment resisting steel frame	8	3	5.5	NR	NR	NR	NR	NR
Special moment resisting steel bar frame	7	3	5.5	NR	NR	48	30	NA
Medium moment resisting steel frame	4.5	3	4	NR	NR	10	NA	NA
Ordinary moment resisting steel frame	3.5	3	3	NR	NR	NA	NA	NA
Special moment resisting reinforced concrete frame	8	3	5.5	NR	NR	NR	NR	NR
Medium moment resisting reinforced concrete frame	5	3	4.5	NR	NR	NA	NA	NA
Ordinary moment resisting reinforced concrete frame	3	3	2.5	NR	NA	NA	NA	NA

Table 3.6 (Continuing) Factor *R*, Ω, and C_d for Seismic Force Resisting System

Seismic Force Resisting System	Response Modification Response	System Overstrength Factor	Deflection Amplification Factor	Structure System limitation and Structural height limitation, h_n (m)				
				Seismic Design Category				
Special moment resisting composite steel and concrete frame	8	3	5.5	NR	NR	NR	NR	NR
Medium moment resisting composite steel and concrete frame	5	3	4.5	NR	NR	NA	NA	NA
Partially restrained moment resisting composite steel and concrete frame	6	3	5.5	48	48	30	NA	NA
Ordinary moment resisting composite steel and concrete frame	3	3	2.5	NR	NA	NA	NA	NA
Special moment resisting cold-rolled steel frame with bolting	3.5	3	3.5	10	10	10	10	10

Source: (BSN, 2019a)

Note: NR = Not Restricted and NA = Not Allowed

3.4 Equivalent Lateral Force Analysis

Planning the structure in Indonesia should consider the earthquake load. Lateral force will occur due to the earthquake load. The equivalent lateral force (ELF) procedure is the method that used to analyse the effect of earthquake's dynamic loading. The planning refers to ASCE/SEI 7-22 (ASCE, 2021).

3.4.1 Seismic base shear

The seismic base shear with force, in a given direction can be determined in accordance with the equation as follow.

$$V = C_s W \tag{3.8}$$

Where:

C_s = The seismic response coefficient determined in accordance with the parameter in section 3.2.2

W = Effective structure mass (kN)

3.4.2 Seismic response coefficient

The seismic response coefficient is influenced by the condition of earthquake area, fundamental period (*T*) of the building, and local soil condition. The equation is determined in accordance as follows:

$$C_{s1} \quad = \frac{S_{DS}}{\left(\frac{R}{Ie}\right)} \tag{3.9}$$

with

S_{DS} = Design spectral response acceleration parameter in the short-range periods,

R = Response modification factor, and

I_e = Importance building factor.

The value of C_{s2} computed with equation (3.10) should not exceed the boundaries as follows:

for $T \leq T_L$,

$$C_{s2} \quad = \frac{S_{D1}}{T\left(\frac{R}{Ie}\right)} \tag{3.10}$$

And for $T > T_L$,

$$C_{s2} \quad = \frac{S_{DS}}{\left(\frac{R}{Ie}\right)} \tag{3.11}$$

With

S_{D1} = Design spectral response acceleration parameters at a period,

T = Fundamental period of the structure(s), and

T_L = Long period transition period.

The value of C_s should not less than the lower boundaries as follows:

$$C_{s\,min} \quad = 0{,}044 S_{DS} I_e \geq 0{,}01 \tag{3.12}$$

3.4.3 Fundamental period and Structure period

Approximate fundamental period (T_a) is the one fully cycle time of building vibration in seconds, can be determined from the equation (3.13).

$$T_a \quad = C_t h_n^x \tag{3.13}$$

where h_n is the structure height and the coefficients C_t and x are determined in Table 3.7.

Table 3.7 Approach Period Parameter Values *Ct* and *x*

Structure Type	C_t	x
Moment-resisting frame system that the frame resists 100% of the seismic forces required and not adjoined by component that have more rigidity:		
Steel moment-resisting frames	0,0724	0,8
Concrete moment-resisting frames	0,0466	0,9
Steel eccentrically braced frame	0,0731	0,75
Steel buckling-restrained braced frames	0,0731	0,75
All other structural systems	0,0488	0,75

(Source: (ASCE, 2021), (BSN, 2019a)))

While Natural period is the structure vibration with single of freedom in accordance with (Paulay & Priestley, 1992) can be used to estimate the stiffness of the structure. The natural period can be determined in accordance with equation (3.14)

$$T = 2\Pi\sqrt{\frac{m}{k\times g}} \tag{3.14}$$

3.4.4 Vertical distribution of seismic forces

The lateral seismic force (F_x) applied at any level of the structure. The calculation can be determined from equation as follows:

$$F_{x_s} = C_{vx}V \tag{3.15}$$

$$C_{vx} = \frac{\mathrm{w_x} h_x^k}{\sum_n^{i=1} w_i h_i^k} \tag{3.16}$$

Where:

C_{vx} = Vertical distribution factor,

V = Structure shear base (ton),

w_i, w_x = Portion of total structure effective weight of the structure (W) that assigned to level i or x,

h_i, h_x = Height from the structure base to level i or x, and

k = Exponent of structure period relation as follows:

for structure period of 0,5 s or less, $k = 1$;

for structure period of 2,5 s or more, $k = 2$; and

for structure period between 0,5 and 2,5 s, k will be determined by linear interpolation between 1 and 2.

3.5 Lateral Force Deformation Method

Ratio of cumulative storey lateral force to inter-storey lateral displacement of design earthquake load structural analysis result is used to estimate storey stiffness(Vijayanarayanan et al., 2017).

One unit of forces is applied to weak axis direction and strong axis direction of the structure. As a result, the deformation of structural analysis is obtained to estimate the storey stiffness. The illustration and the equation of the method can be seen in Figure 3.5.

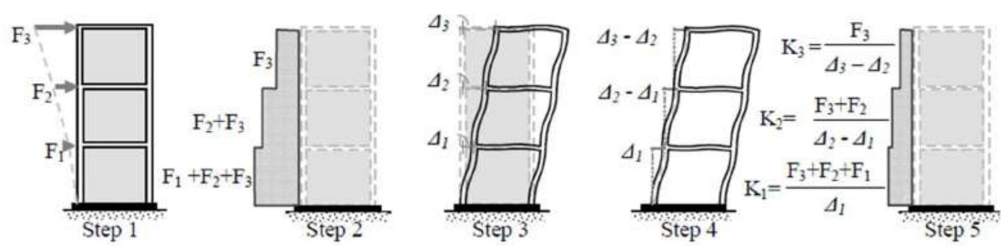

Figure 3.5 Lateral Force Deformation Method

(Source: Vijayanarayanan et al., 2017)

3.6 Structural Irregularities

Structural configuration is divided into two aspects namely horizontal and vertical configuration. The structure configuration performance can significantly be affected during the ground motion contemplated in the standard by the strong earthquake. The irregular configuration building suffers greater damage due to earthquake.

The reason for the poor performance of irregular structure is the irregular structure inelastic behaviour possibly concentrated by irregularities and resulting in rapid failure of structural element in irregularity areas (ASCE, 2021). The with irregularities should analysed with allowed analysis procedure in accordance with Table 3.8.

Table 3.8 Allowed Analysis Procedure

Seismic Design Category	Structure Characteristic	Equivalent Lateral Force Analysis	Modal Response Spectrum	Linear Response History Analysis
B, C	All Structure	A	A	A
D, E, F	Buildings with risk category I or II that do not exceed 2 storeys above the ground.	A	A	A
	Structures without structural irregularities and whose height does not exceed 48.8 m	A	A	A
	Structures without structural irregularities with a height exceeding 48.8 m and T < 3.5 Ts	A	A	A
	Structures with a height not exceeding 48.8 m and having only horizontal irregularities of type 2, 3, 4 or 5 or vertical irregularities of type 4, 5a or 5b	A	A	A
	All other structures	P	A	A

(Source: (BSN, 2019a))

Note: A: Allowed, P: Prohibited

3.6.1 Horizontal Irregularity

The structure is designate as horizontal structural irregularity when the structures have one or more of the horizontal irregularity types listed on the Table 3.9.

Torsional Irregularity Ratio (*TIR*) is the calculation for the accidental torsion case and each building story. The calculation includes the maximum story drift at the building's edge and the average of the story drifts at the two opposite edge of the building. The lateral forces using equivalent lateral force procedure. The formula of the *TIR* in accordance with equation (3.17)

$$TIR = \frac{\Delta_{max}}{\Delta_{avg}} \tag{3.17}$$

Table 3.9 Horizontal Irregularity Classification

Type	Description	Article Reference	Application Seismic Design Category
1a	Torsional Irregularity: Torsional irregularity, defined where the Torsional Irregularity Ratio (TIR) more than 1.2	7.3.3.4 7.7.3 7.8.4.3 7.12.1 Table 16 11.3.4	D, E, F B, C, D, E, F C, D, E, F C, D, E, F D, E, F B, C, D, E, F

Table 3.9 (Continuing) Horizontal Irregularity Classification

Type	Description	Article Reference	Application Seismic Design Category
1b	Excessive Torsional Irregularity: Excessive Torsional irregularity, defined where the Torsional Irregularity Ratio (TIR) more than 1.4	7.3.3.1 7.3.3.4 7.3.4.2 7.7.3 7.8.4.3 7.12.1 Table 16 11.3.4	E, F D B, C, D C, D C, D D B, C, D
2	An inside corner irregularity is defined to exist if both the dimensions of the structure plan projection of the inside corner location are greater than the than 15% of the structure plan dimension in the direction under review.	7.3.3.4 Table 16	D, E, F D, E, F
3	A diaphragm discontinuity irregularity is defined to exist if there is a diaphragm that has discontinuities or abrupt variations of stiffness, including those with a truncated or open area greater than 50% of the gross diaphragm area. or open area greater than 50 % of the gross diaphragm area that is area, or a change in effective diaphragm stiffness of more than 50%. from one level to the next.	7.3.3.4 Table 16	D, E, F D, E, F
4	Irregularity due to displacement perpendicular to the plane is defined to exist if there is a discontinuity in the trajectory of the plane are defined to exist if there is a discontinuity in the path of the lateral force resistance, such as displacement perpendicular to the plane in at least one lateral force-bearing vertical element.	7.3.3.3 7.3.3.4 7.7.3 Table 16 11.3.4	B, C, D, E, F D, E, F B, C, D, E, F D, E, F B, C, D, E, F
5	A nonparallel system irregularity is defined to exist if the lateral force-bearing vertical elements are not parallel to the axis the main orthogonal axis of the seismic force-carrying system.	7.5.3 7.7.3 Table 16 11.3.4	C, D, E, F B, C, D, E, F D, E, F B, C, D, E, F

(Source: (BSN, 2019a))

The illustration of the horizontal irregularity types is served to illustrate the classification types. The illustration is listed as follows.

1. Horizontal Irregularity Type 1a and 1b

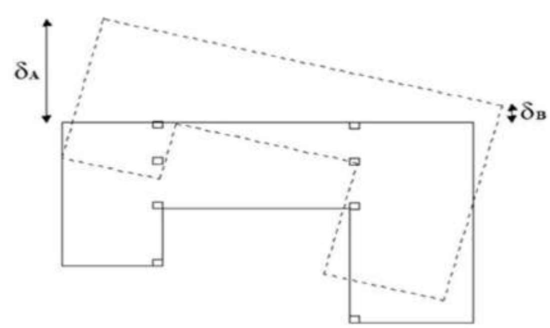

Figure 3.6 Torsional Irregularity Illustration

(Source: (BSN, 2019a))

2. Horizontal Irregularity Type 2

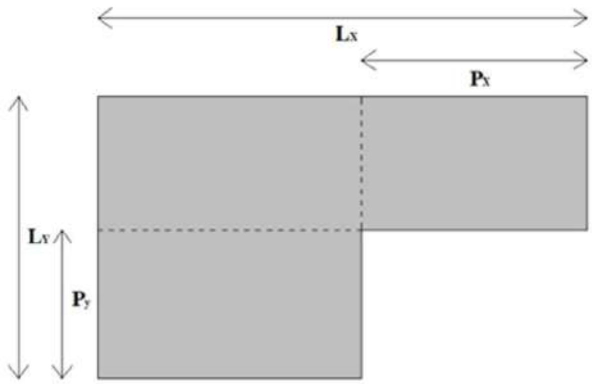

Figure 3.7 An Inside Corner Irregularity Illustration

(Source: (BSN, 2019a))

3. Horizontal Irregularity Type 3

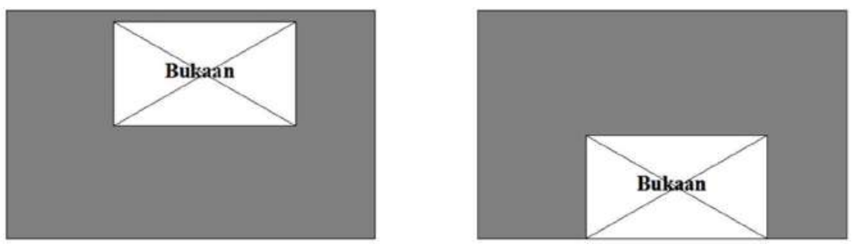

Figure 3.8 A Diaphragm Discontinuity Irregularity Illustration

(Source: (BSN, 2019a))

4. Horizontal Irregularity Type 4

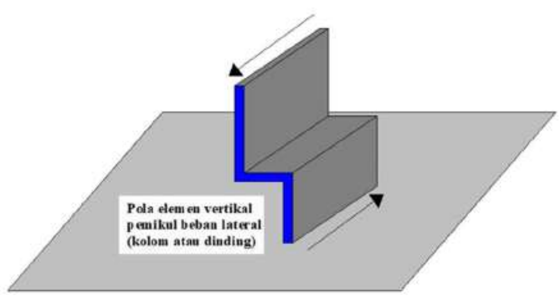

Figure 3.9 Irregularity Due to Displacement Perpendicular to the Plane Illustration

(Source: (BSN, 2019a))

5. Horizontal Irregularity Type 5

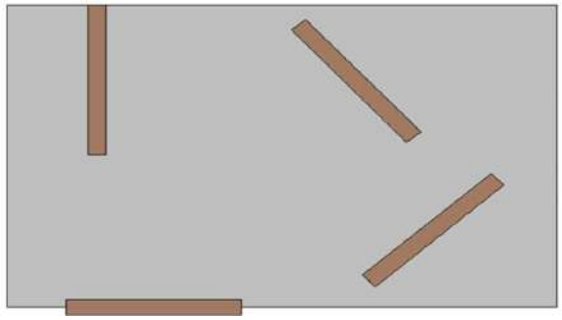

Figure 3.10 A Nonparallel System Irregularity Illustration

(Source: (BSN, 2019a))

3.6.2 Vertical Irregularity

The structure is designate as vertical structural irregularity when the structures have one or more of the vertical irregularity types listed on the Table 3.10.

Table 3.10 Vertical Irregularity Classification

Type	Description	Article Reference	Application Seismic Design Category
1a	Soft Level Stiffness Irregularity is defined as existing if there is a level whose lateral stiffness is less than 70% of the lateral stiffness of the level above it or less than 80% of the lateral stiffness of the level above it. the average stiffness of the three levels above it.	Table 16	D, E, F
1b	Excessive Soft Level Stiffness Irregularity defined as existing if there is a level whose lateral stiffness lateral stiffness is less than 60% of the lateral stiffness of the level above it or less than 70% of the average stiffness of the three levels above.	7.3.3.1 Table 16	E, F D, E, F
2	Weight (Mass) Irregularity is defined to exist if the effective mass at any level is more than 150 % of the effective mass of the neighbouring level. Roofs that are lighter than the floor below do not need to be reviewed.	Table 16	D, E, F
3	Vertical Geometry Irregularity is defined as existing if the horizontal dimension of the seismic force resisting system at any level is more than 130 % of the horizontal dimension of the seismic force resisting system of the adjacent level.	Table 16	D, E, F

Table 3.6 (Continuing) Vertical Irregularity Classification

Type	Description	Article Reference	Application Seismic Design Category
4	Irregularity Due to Plane Discontinuity in Lateral Force Bearing Vertical Elements is defined to exist if the displacement of the plane direction of a lateral force bearing element is greater than the length of that element or there is a reduction in the stiffness of the bearing element at the level below.	7.3.3.3 7.3.3.4 Table 16	B, C, D, E, F D, E, F D, E, F
5a	Weak Level Irregularity due to Discontinuity in Level Lateral Strength is defined as existing if the lateral strength of a level is less than 80% of the lateral strength of the level above. The lateral strength of a level is the total strength of all seismic bearing elements sharing the level shear in the direction under review.	7.3.3.1 Table 16	E, F D, E, F
5b	Excessive Weak Level Irregularity Due to Discontinuity in Level Lateral Strength is defined as existing if the lateral strength of a level is less than 65 % of the lateral strength of the level above. The lateral strength of a level is the total strength of all seismic bearing elements sharing the level shear in the direction under review.	7.3.3.1 7.3.3.2 Table 16	D, E, F B, C D, E, F

(Source: (BSN, 2019a))

The illustration of the vertical irregularity types is served to illustrate the classification types. The illustration is as follows:

1. Vertical Irregularity Type 1a and 1b

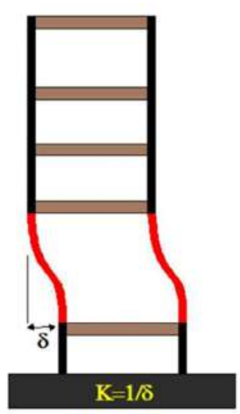

Figure 3.11 Soft and Excessive Soft Level Stiffness Irregularity Illustration

(Source: (BSN, 2019a))

2. Vertical Irregularity Type 2

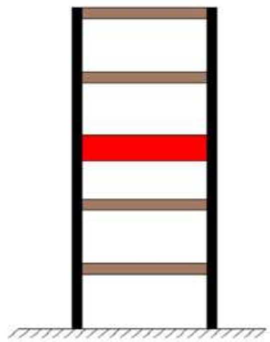

Figure 3.12 Weight (Mass) Irregularity Illustration

(Source: (BSN, 2019a))

3. Vertical Irregularity Type 3

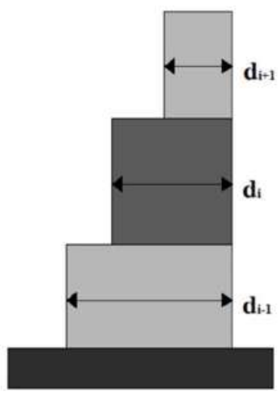

Figure 3.13 Vertical Geometry Irregularity Illustration

(Source: (BSN, 2019a))

4. Vertical Irregularity Type 4

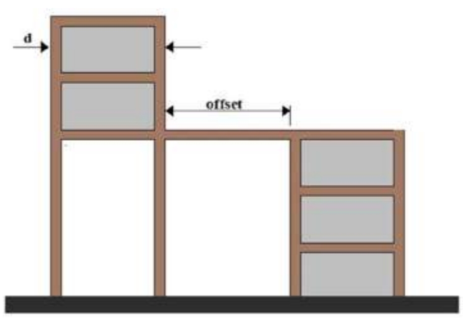

Figure 3.14 Irregularity Due to Plane Discontinuity in Lateral Force Bearing Vertical Elements Illustration

(Source: (BSN, 2019a))

5. Vertical Irregularity Type 5a and 5b

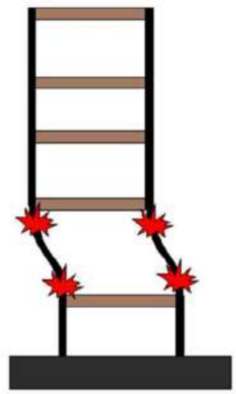

Figure 3.15 Weak and Excessive Weak Level Irregularity Illustration
(Source: (BSN, 2019a))

3.7 Inelastic Response

The earthquake ground shaking naturally produce large portion of the shaking energy that can be dissipated by inelastic deformations when the structure is ductile and acceptable structure damage (National Institute of Building Sciences Building Seismic Safety Council, 2012). Since the analysis of the structure is reduced by *R* when calculate the seismic response coefficient (*Cs*), the displacement is representative of an elastic system. The peak displacement of inelastic system tends to exceed elastic system displacement. Hence the elastic system displacement is amplified with amplification factor, Cd to account for inelastic system and applies an upper limit period or structure period that exceed the upper limit to achieve consistent drifts (ASCE, 2021). The amplification for elastic system to account for inelastic system is in accordance with equation (3.18)

$$\delta_x = \frac{C_d \delta_{xe}}{I_e} \tag{3.18}$$

3.8 Story Drift

The story drift is limited because to address the structural performance of member inelastic strain and stability of the system and to limit the non-structural component damage, which possibly life-threatening (ASCE, 2021). The drift is related to the P-delta effect when under small lateral deformations condition, the P-delta effect is tolerable.

The designed story drift is taken from the design earthquake displacement at the centre of mass (CoM) at each story. The drift is computed in according to the Equivalent Lateral Force or Modal Response Spectrum or linear response history analysis procedure, the earthquake force procedure is in accordance with the structure requirement and specification.

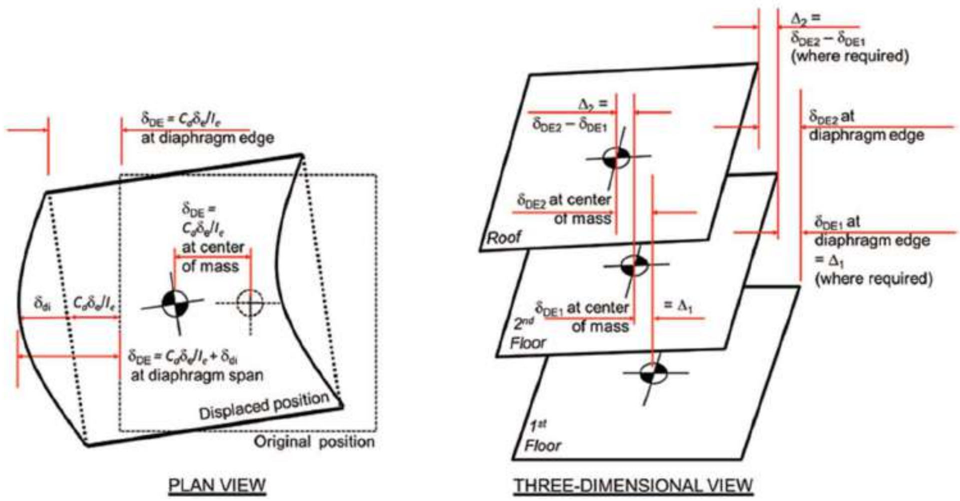

Figure 3.16 Design Earthquake Displacement and Design Story Drift

Source: (ASCE, 2021)

The computation for the drift analysed in inelastic system is in accordance with equation (3.18). The story drift design cannot exceed allowable drift in accordance with Table 3.11.

Table 3.11 Allowable Story Drift

Structure	Risk Category		
	I or II	III	IV
A structure, other than a masonry shear wall structure, of 4 stories or less with interior walls, partitions, ceilings, and an exterior wall system designed to accommodate the displacement between stories.	$0.025h_t$	$0.02h_t$	$0.015h_t$
Masonry cantilever shear wall structure	$0.010h_t$	$0.010h_t$	$0.010h_t$
Other masonry shear wall structures	$0.007h_t$	$0.007h_t$	$0.007h_t$
All other structure	$0.020h_t$	$0.015h_t$	$0.010h_t$

(Source: (BSN, 2019a))

Note: h_t is the story height below reviewed story.

3.9 P-delta Effect

The P-delta effect occurs when the top of the structure displaces laterally, the gravity load, the axial load, supported by the structure causing displacement and produce an overturning moment. The displacement for the P-delta effects can be taken from the displacement on the story drift design.

P-delta effects on story shears and structure moments, causing structure member forces and moments, when the computed stability coefficient is equal to or less than 0.10, the story drift and the P-delta effect are not required to be considered. While the stability coefficient more than 0.10 the stability coefficient cannot exceed maximum stability coefficient(ASCE, 2021). The formula of the stability coefficient is in accordance with equation (3.19) and the maximum stability coefficient is in accordance with equation (3.20).

$$\theta = \frac{\mathrm{P_x} \Delta I_e}{V_x h_{sx} C_d} \tag{3.19}$$

$$\theta_{max} = \frac{0.5}{\beta\ C_d} \leq 0.25 \tag{3.20}$$

3.10 Beam Element

Beam design considering the ultimate strength, where the ultimate strength is the strength that will occur in the real site. Ultimate strength multiplied with safety factor to reduce the planned strength. The planned strength should reduce but should be more than ultimate strength to reach the safe structure. The beam design is in accordance with SNI 2847-2019 (BSN, 2019b)

3.10.1 Beam dimension limitation

Non-prestressed beam that does not supported or attached to partition or other construction likely can damaged by large deflection; the height of the beam h_b will satisfy the limits in accordance with Table 3.12.

Table 3.12 Minimum *h* for Beam Supporting Deflection-insensitive Non-structural Element

Condition	Minimum h_b
Simple support	$\frac{l}{16}$
One end continuous	$\frac{l}{18,5}$
Both end continuous	$\frac{l}{21}$
Cantilever	$\frac{l}{8}$

Source: (BSN, 2019b)

Equation in Table 3.2 can be applied for the normal concrete quality and steel quality (f_y) of 420 MPa. For the steel quality 280 MPa $\leq f_y \leq$ 550 MPa, the equation from Table 3.2 should be multiplied with (0,7 + f_y /700).

The design limitation for the beam that carry the deflection-sensitive non-structural elements such as the glass or other brittle materials that will be damaged by deflections, the depth of beams should not less than the depth in accordance with Table 3.13.

Table 3.13 Minimum *h* for Beam Supporting Deflection-sensitive Non-Structural Element

Condition	Minimum h_b
Simple support	$\frac{l}{11}$
One end continuous	$\frac{l}{12}$
Both end continuous	$\frac{l}{14}$
Cantilever	$\frac{l}{5}$

(Source: (ACI Committee 314, 2016))

CHAPTER IV
RESEARCH METHODS

4.1 Introduction

Research methods is the sequence of the scientific steps to obtain data and answers from the formulated problems. Research methods are based on scientific way referring on characteristics of science, i.e. systematically, rationally, and empirically basis.

4.2 Research Object

The object of this research is 7 stories hotel building with special moment reinforcement concrete frame that located in Ngawi district, East Java. The hotel building plan will be modified by applying 5 secondary beam configurations. Thus, there are 5 structure model objects.

4.3 Research Data

Data of the research consists of three data. The data are seismic data, structural data, and materials data for the design. Seismic data is in accordance with the earthquake resistant building code value of material data for this research will be determined according to the general material. While the structural data obtains from 3-dimensional software analysis for reinforcement concrete frame reaction.

4.4 Material and Loading

The materials and loadings in this research define as follows:

1. Concrete quality (f_c') is 25 MPa and the elastic modulus (E_c) is $4700\sqrt{f_c'}$,
2. Rebars yield strength are 280 MPa for plain bar shear reinforcement and 420 MPa for deformed bar flexural reinforcement, and
3. The earthquake load is analysed procedure in according to SNI 1727:2019 and ASCE 7-22.

4.5 Structure Model

The structures model plan uses for this research is Nata Hotel Ngawi. The frame of the building structure is modified with variation of secondary beams configuration. The building plan can be seen in Figure 4.1.

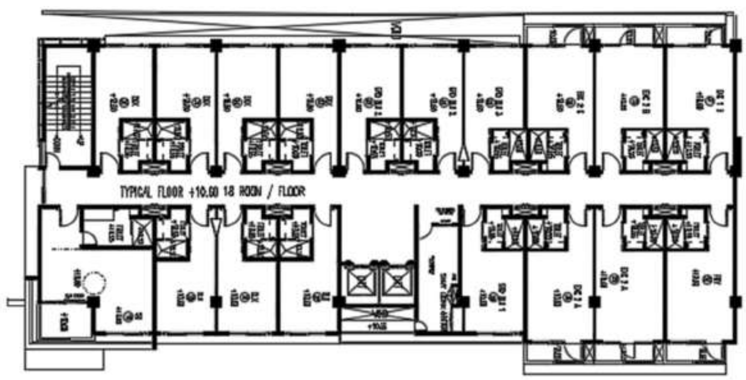

Figure 4.1 Nata Hotel Ngawi Building Plan

This research modifies the building plan in Figure 4.1 by applying the secondary beam and the structure response result will be compared. The secondary beam configuration variations are as follows:

1. Structure without secondary beam on building layout,
2. Structure with 1 secondary beam at weak axis of building layout,
3. Structure with 1 secondary beam at strong axis of building layout,
4. Structure with secondary beams at both axes of building layout, and
5. Structure with secondary beams at both diagonally axes of building layout.

The application of secondary beam variations on Nata Hotel building plan can be seen in Figure 4.2 to Figure 4.6.

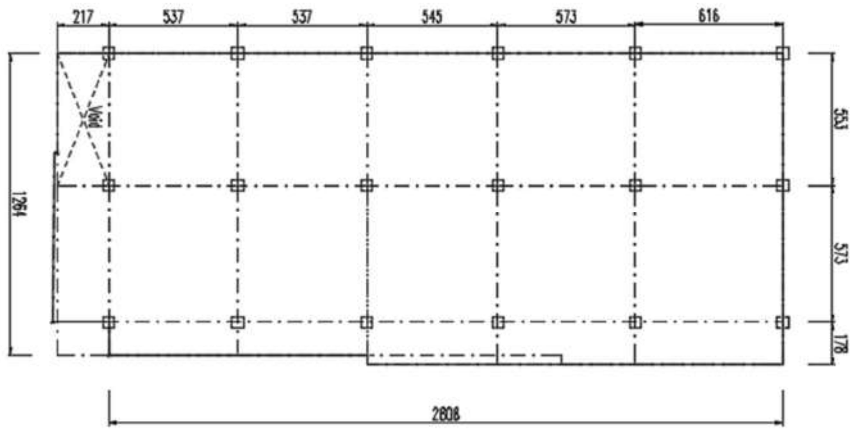

Figure 4.2 Configuration Model 1

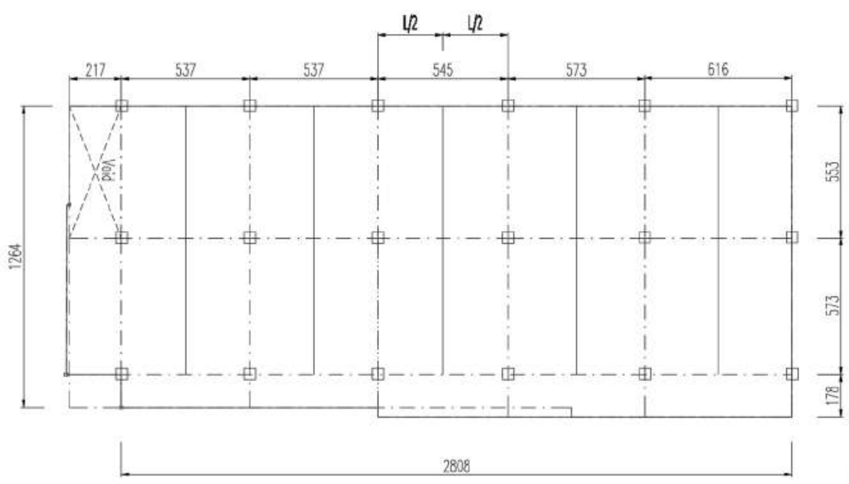

Figure 4.3 Configuration Model 2

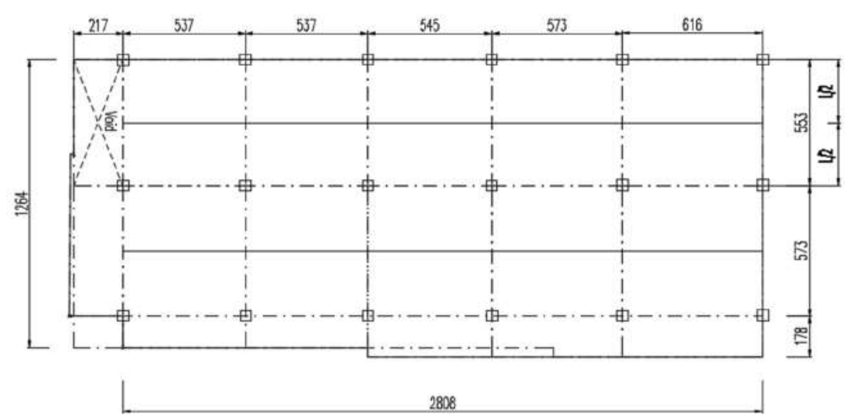

Figure 4.4 Configuration Model 3

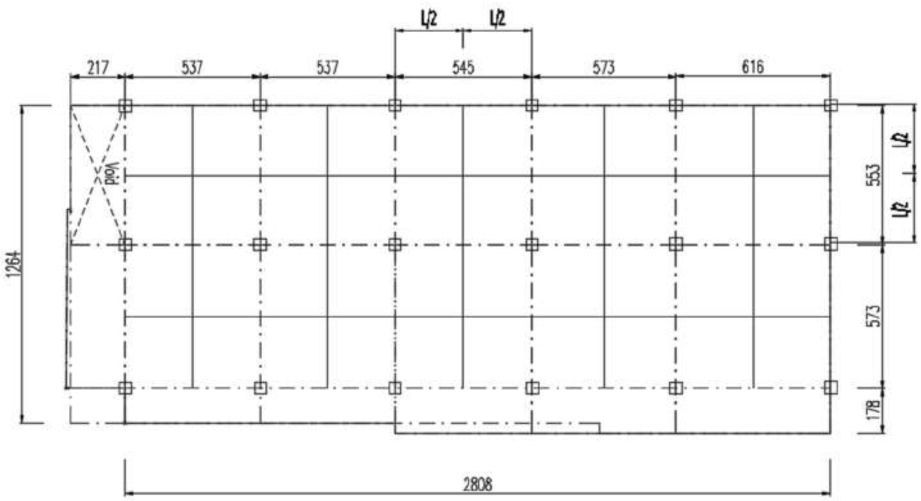

Figure 4.5 Configuration Model 4

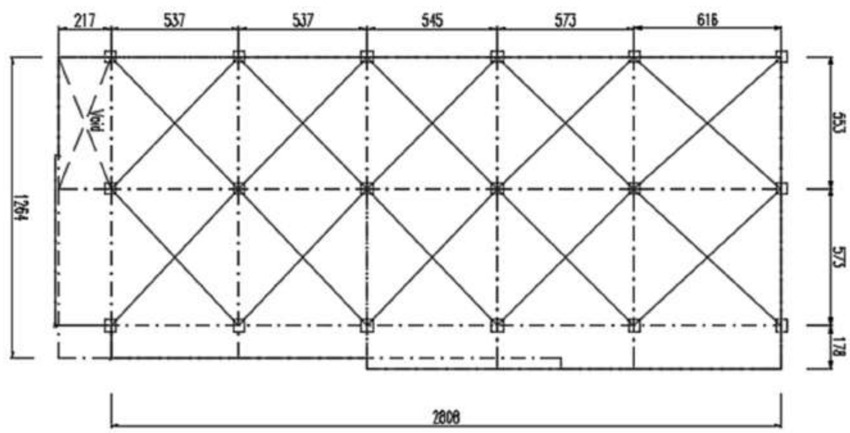

Figure 4.6 Configuration Model 5

4.6 Analysis Procedure

The analysis procedure for this research will conduct as follows.

1. Collect the data

   The data collected including seismic data, material data, and model data. The seismic data is based on research object location, the material data is set by writer to obtain specific and in line result, and model data is Nata Hotel Ngawi, the model data used is only structure plan.

2. Seismic design requirement.

The seismic design requirement is computed initially with Microsoft excel. Determination of seismic forces and analyse the response spectrum referring to the building location.

3. Model configuration

   Arrange the initial structure model with the application of secondary beams variations and determine the structure element preliminary. Calculate the working loads of the structure. The working loads consists of dead loads, live loads, and earthquake loads

4. Data input to SAP2000

   Arrange and input the structure model, material specification, loading, ground motion values, and spectra into software to be analysed 3 dimensionally.

5. SAP2000 data output

   Structure mass, structure period, and structure displacement result from SAP2000 analysis is extracted.

6. Structure response analysis

   The result of SAP2000 analysis is inputted to Microsoft excel to obtain structure response, structure irregularity, story drift, and P-delta effect. The results is checked the structural response accordance to the code requirement. The structural irregularities, story drift, and P-delta effect results from each structure model is compared.

7. Obtain the conclusion from the result of analysis and structure volume.

The flowchart procedure arranges systematically in accordance with Figure 4.7.

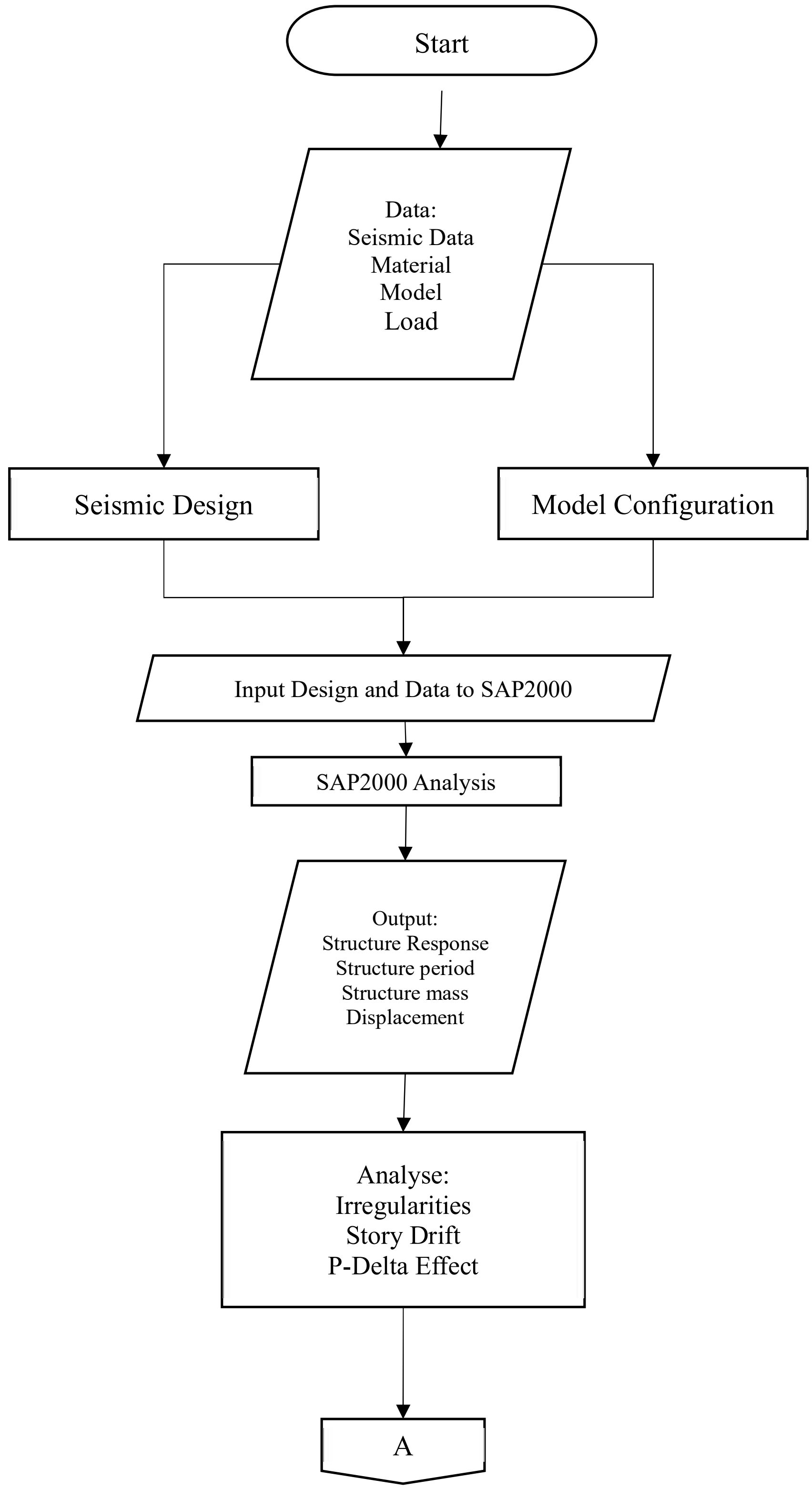

Figure 4.7 Analysis Flowchart

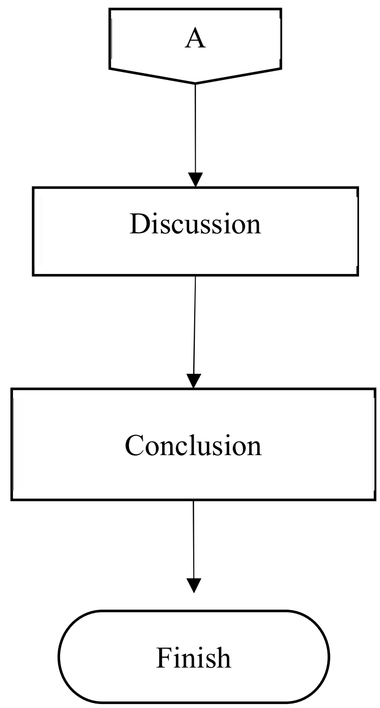

Figure 4.7 (Continuing) Analysis Flowchart

CHAPTER V
ANALYSIS AND DISCUSSION

5.1 Introduction

The analysis of the structure use structural analysis program is controlled and inputted by several manual calculations to ensure the seismic parameter, structural element, and the 3D model in structural analysis program. Since the result of the structural analysis program is dependent on the user input. The building mass and Equivalent Lateral Force analysis will be the control of structural analysis program input. Result of structural analysis will compare to know the influence of secondary beam in the structure.

5.2 Plan and Codification

The structure plan drawing and codification of structure member is presented to serve clear understanding of structure shape. Site plan of building plan as well as story 1 can be seen in the Figure 5.1. While the side view and front view of the building can be seen in Figure 5.2 and Figure 5.3.

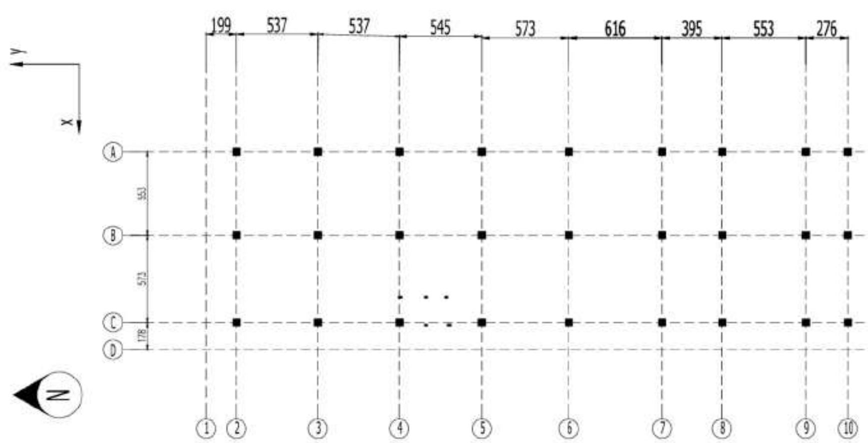

Figure 5.1 Story 1 Plan

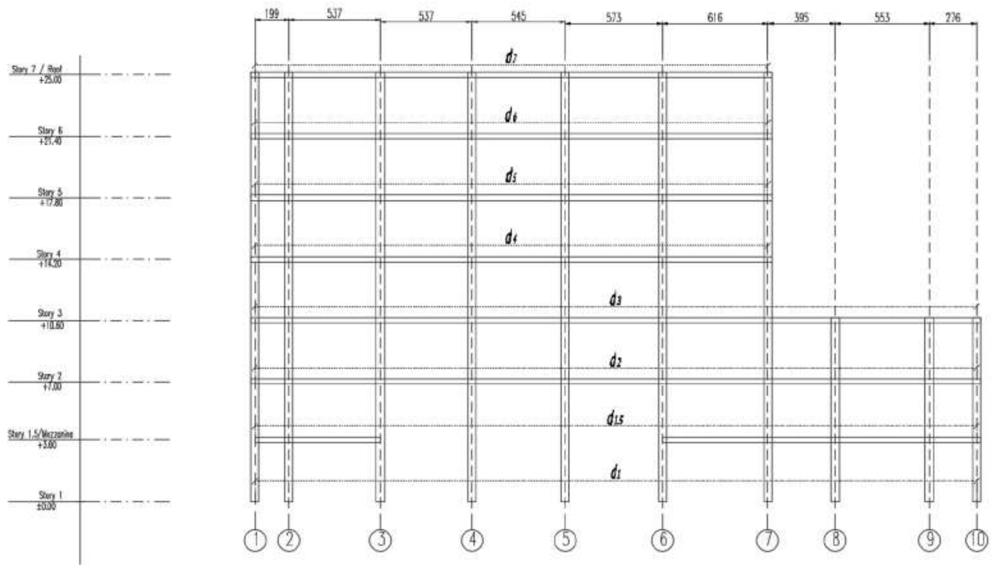

Figure 5.2 Side View Plan

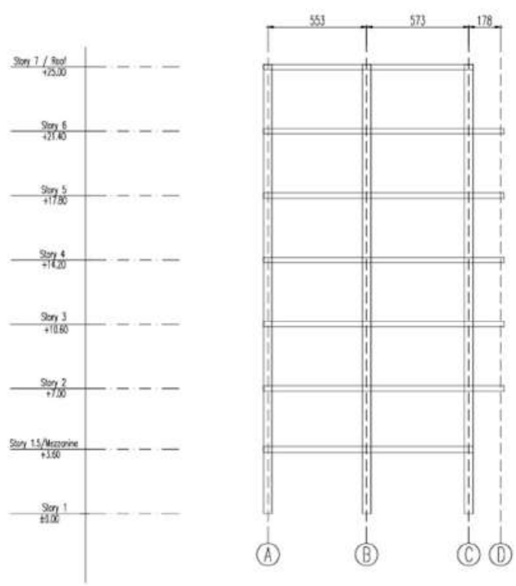

Figure 5.3 Front View Plan

Column member codification of the structure has some different for several story. Hence, the column member codification can be seen in Figure 5.4 to Figure 5.6.

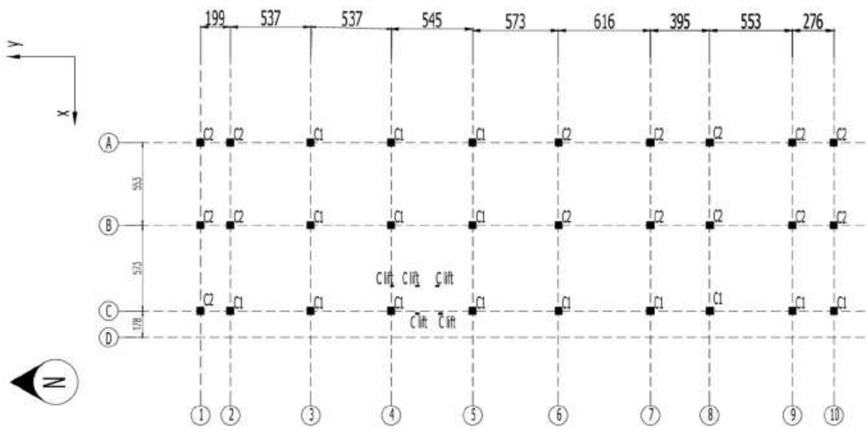

Figure 5.4 Column Codification in Story 1

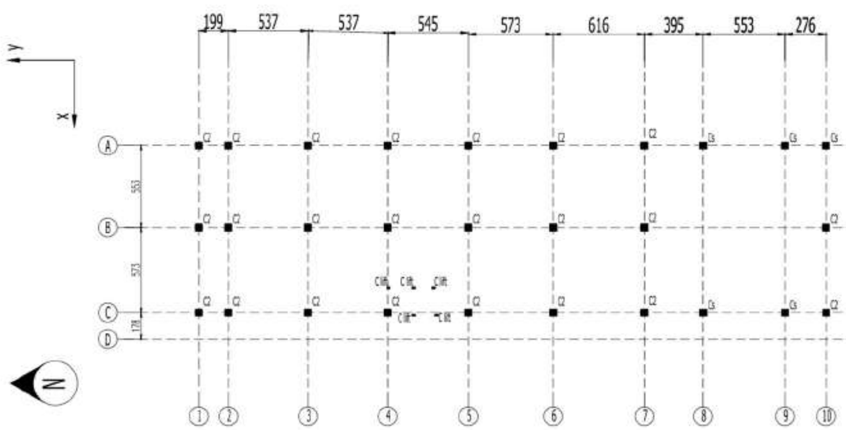

Figure 5.5 Column Codification in Stories 2 to 3

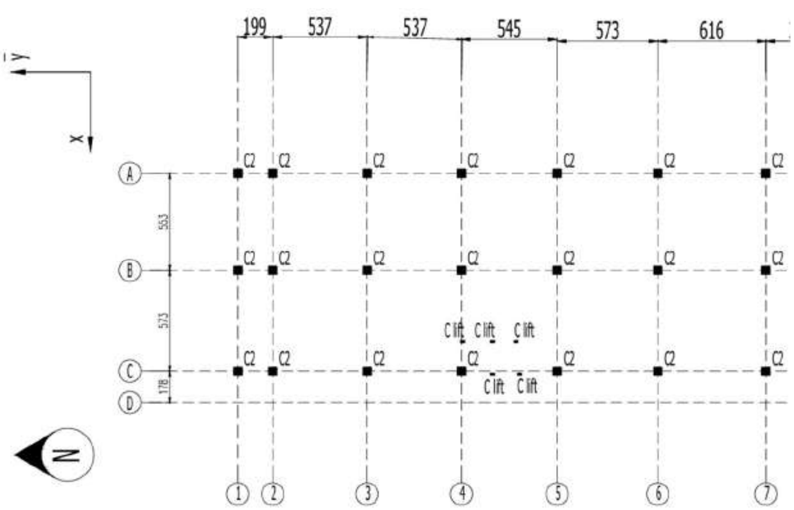

Figure 5.6 Column Codification in Stories 4 to 7

The Figure 5.1 to Figure 5.6 is set same for every model variation in this research. Thus, the other structure member of other story level and other model variations plan is listed and presented in Figure 5.7 to Figure 5.46.

1. Configuration 1
   a. First mezzanine story beam and slab code.

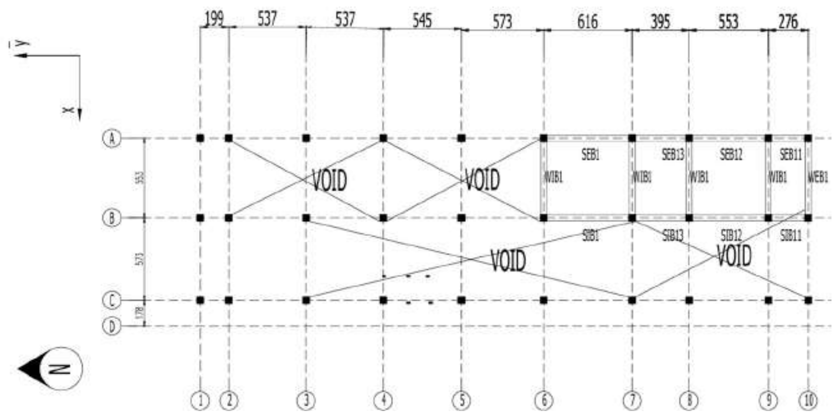

Figure 5.7 Beam Codification on Story 1.5 in Configuration Model 1

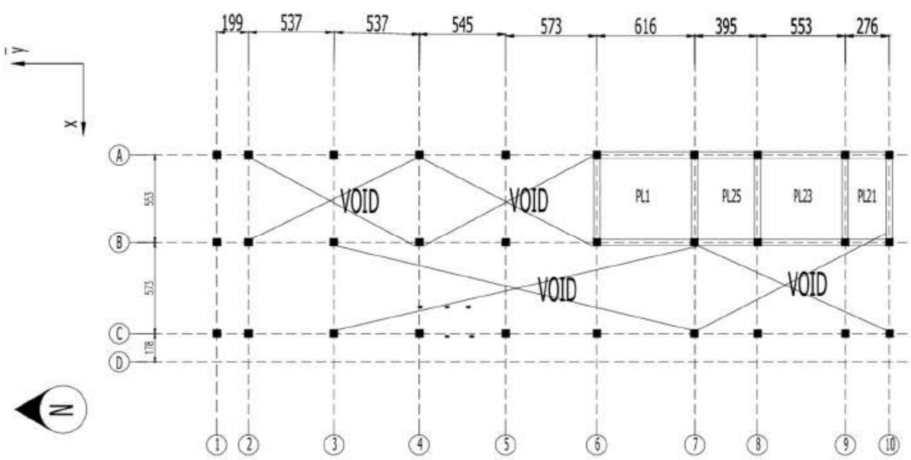

Figure 5.8 Slab Codification on Story 1.5 in Configuration Model 1

b. Second story beam and slab code.

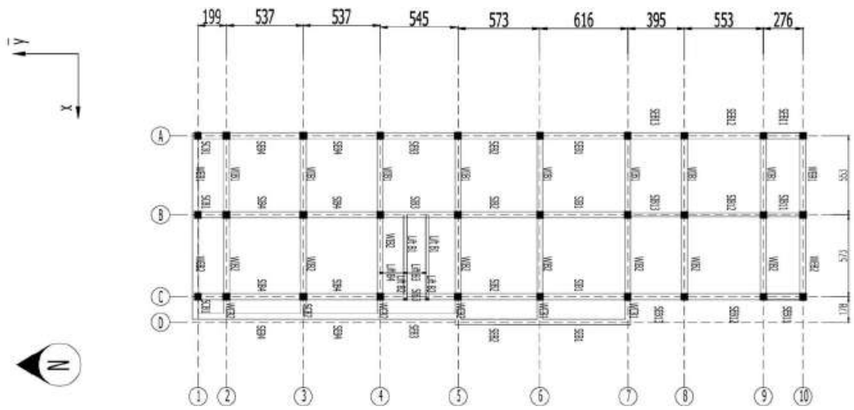

Figure 5.9 Beam Codification on Story 2 in Configuration Model 1

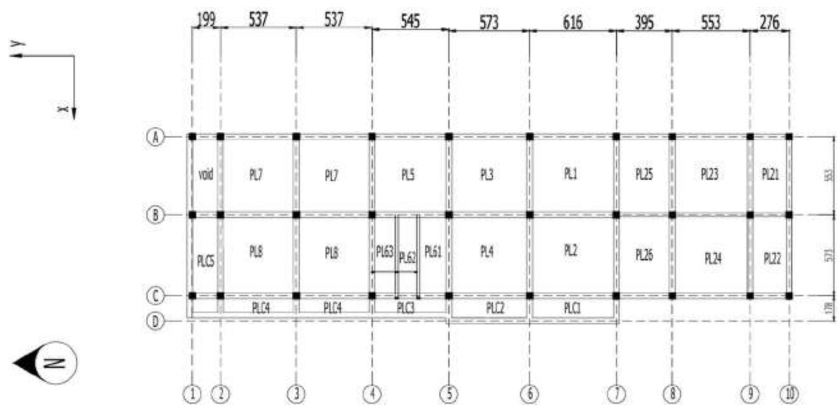

Figure 5.10 Slab Codification on Story 2 in Configuration Model 1

c. Third story beam and slab code.

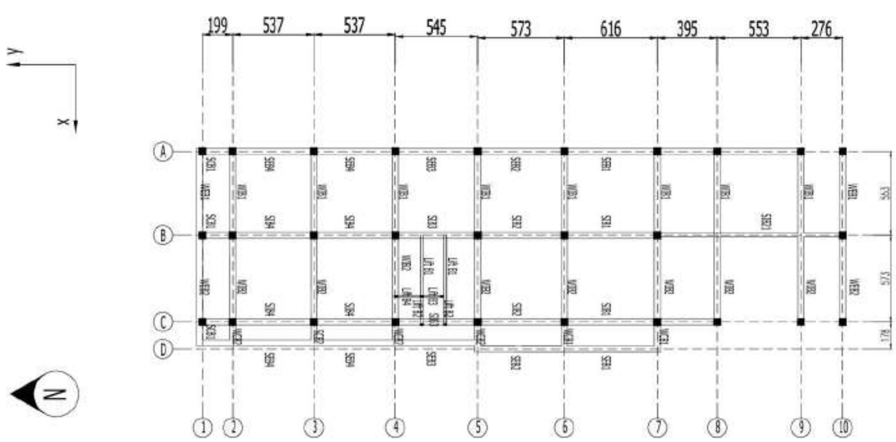

Figure 5.11 Beam Codification on Story 3 in Configuration Model 1

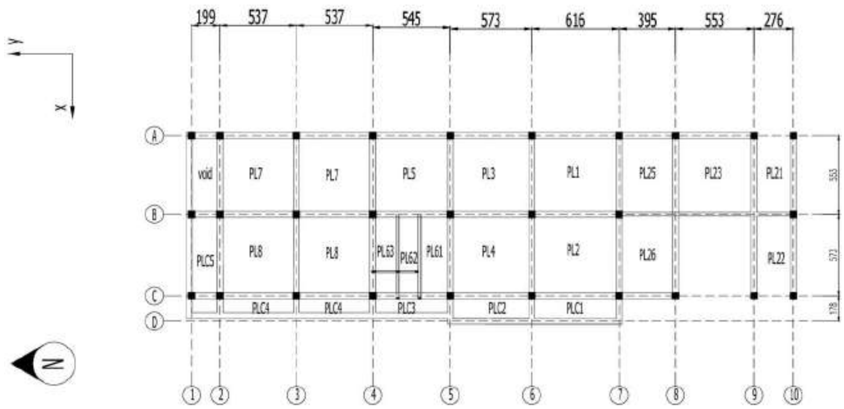

Figure 5.12 Slab Codification on Story 3 in Configuration Model 1

d. Fourth to seventh story beam and slab code.

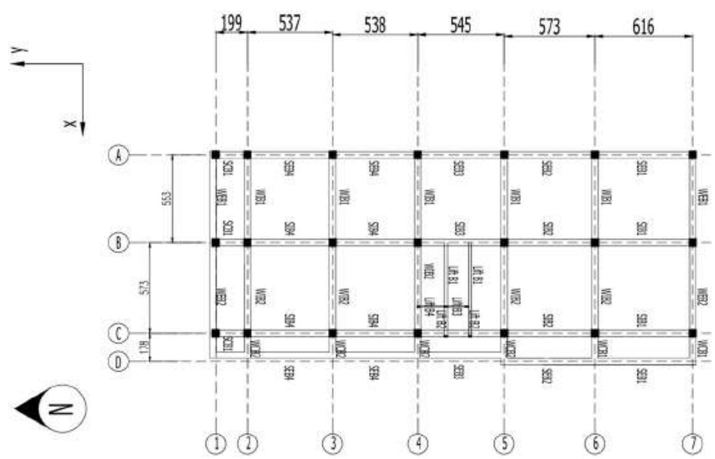

Figure 5.13 Beam Codification on Stories 4 to 7 in Configuration Model 1

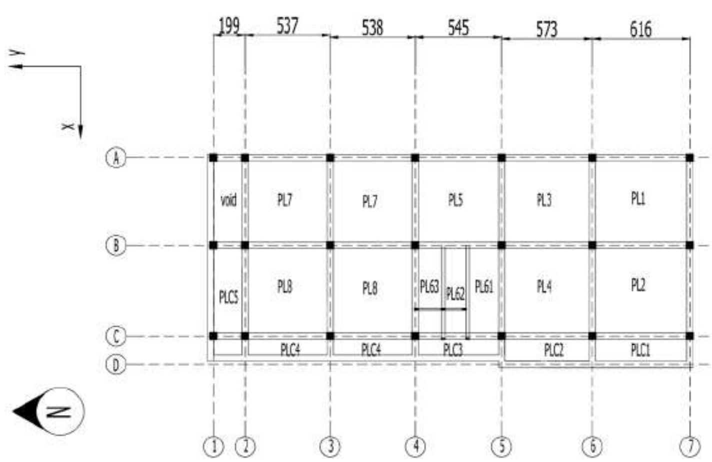

Figure 5.14 Slab Codification on Stories 4 to 7 in Configuration Model

2. Configuration 2

a. First mezzanine story beam and slab code

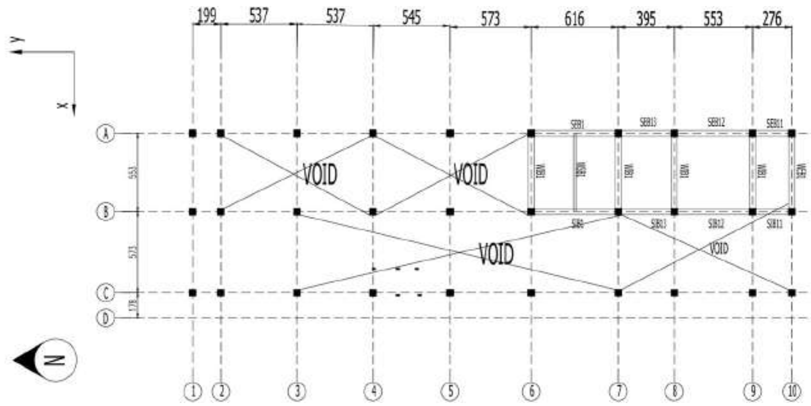

Figure 5.15 Beam Codification on Story 1.5 in Configuration Model 2

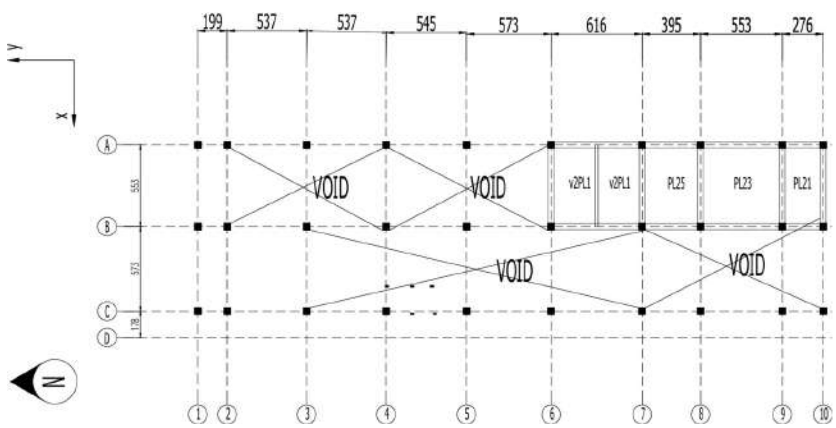

Figure 5.16 Slab Codification on Story 1.5 in Configuration Model 2

b. Second story beam and slab code.

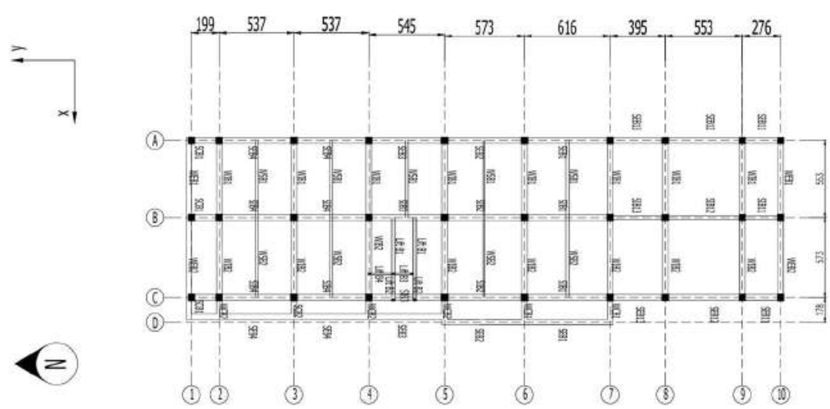

Figure 5.17 Beam Codification on Story 2 in Configuration Model 2

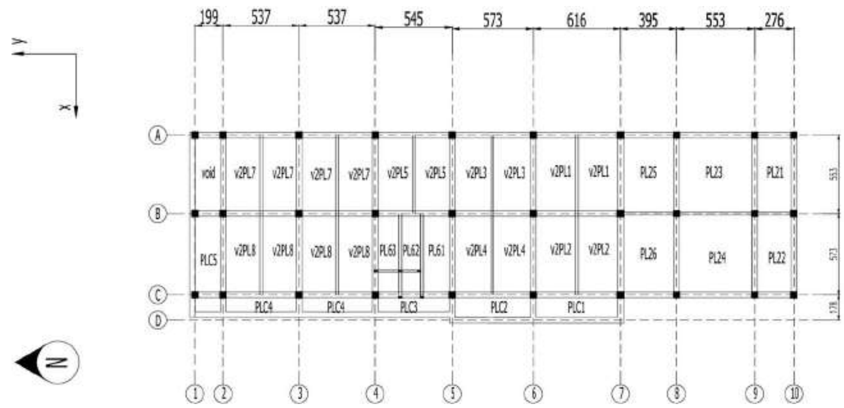

Figure 5.18 Slab Codification on Story 2 in Configuration Model 2

c. Third story beam and slab code.

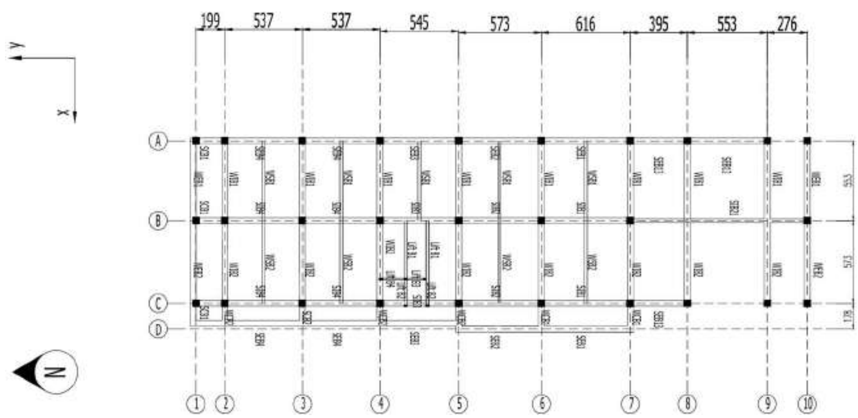

Figure 5.19 Beam Codification on Story 3 in Configuration Model 2

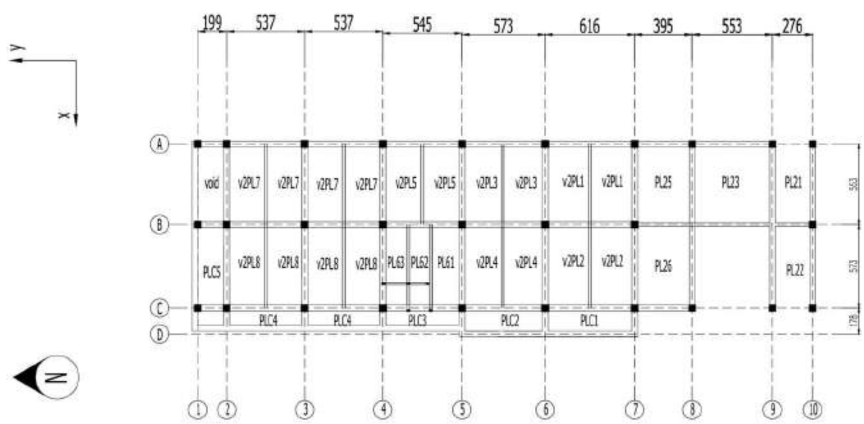

Figure 5.20 Slab Codification on Story 3 in Configuration Model 2

d. Fourth to seventh story beam and slab code.

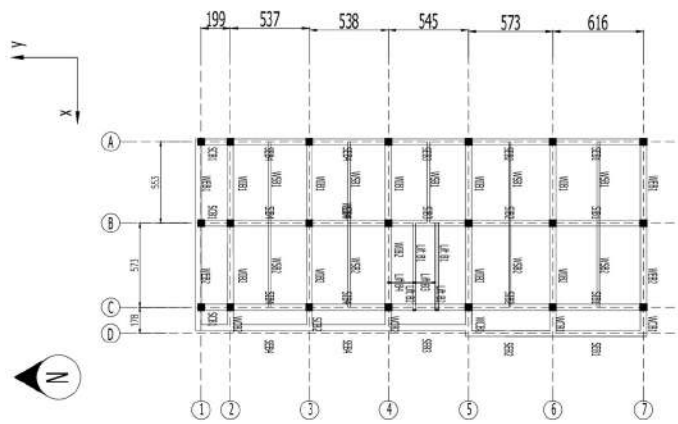

Figure 5.21 Beam Codification on Stories 4 to 7 in Configuration Model 2

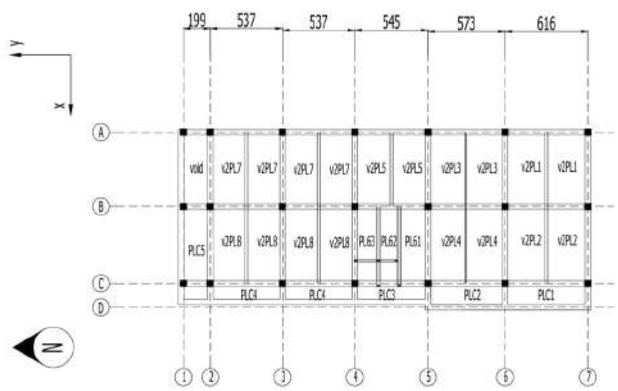

Figure 5.22 Slab Codification on Stories 4 to 7 in Configuration Model 2

3. Configuration 3

   a. First mezzanine story beam and slab code.

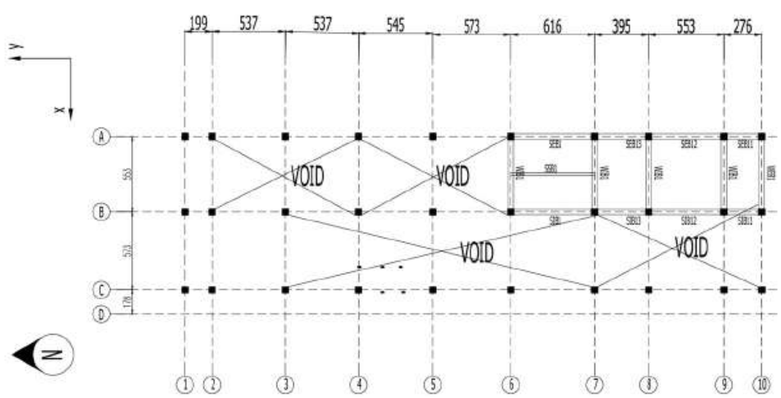

Figure 5.23 Beam Codification on Story 1.5 in Configuration Model 3

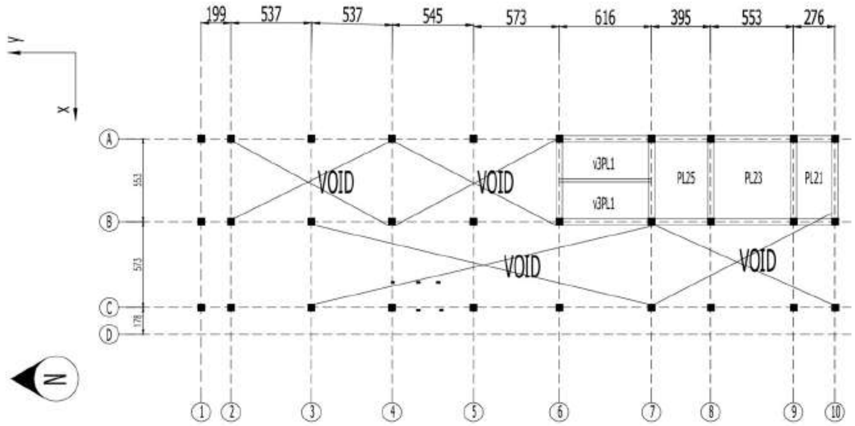

Figure 5.24 Slab Codification on Story 1.5 in Configuration Model 3

b. Second story beam and slab code.

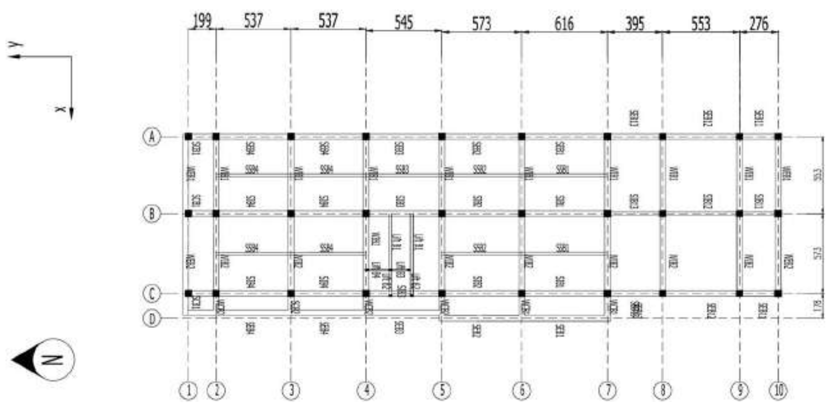

Figure 5.25 Beam Codification on Story 2 in Configuration Model 3

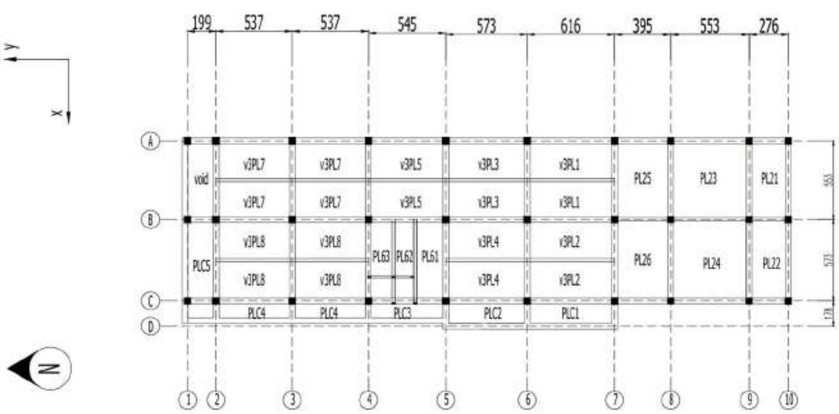

Figure 5.26 Slab Codification on Story 2 in Configuration Model 3

c. Third story beam and slab code.

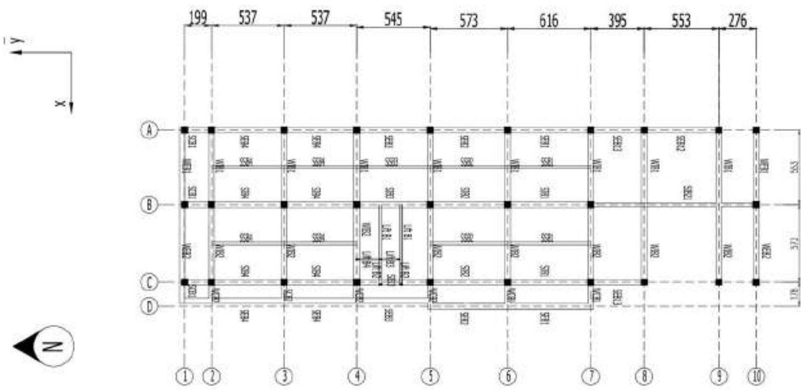

Figure 5.27 Beam Codification on Story 3 in Configuration Model 3

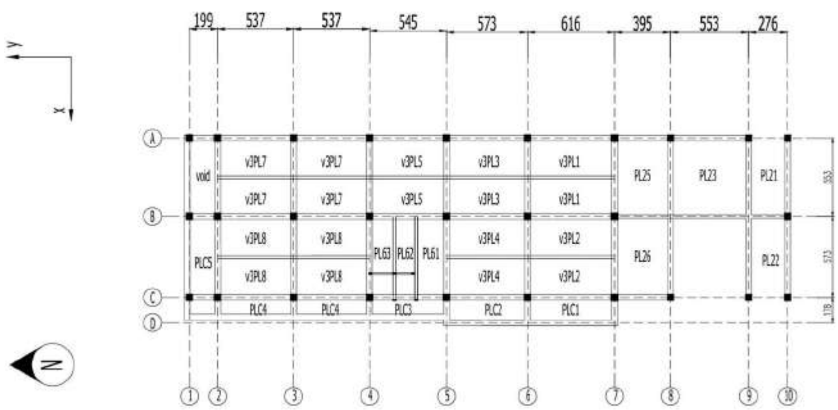

Figure 5.28 Slab Codification on Story 3 in Configuration Model 3

d. Fourth to seventh story beam and slab code.

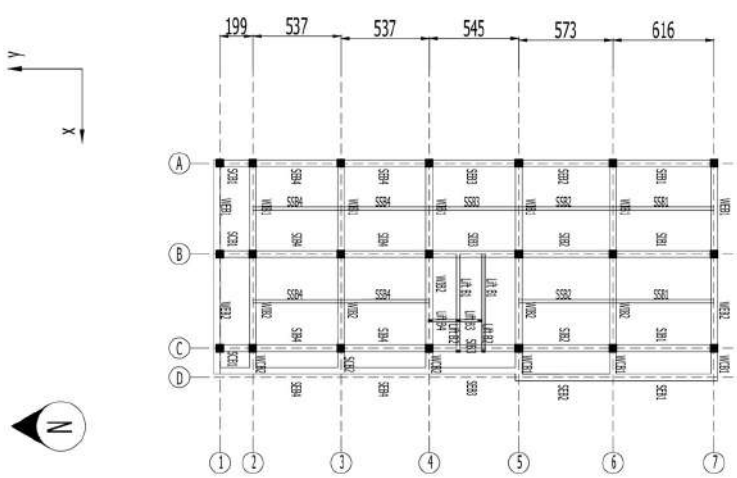

Figure 5.29 Beam Codification on Story 4 to 7 in Configuration Model 3

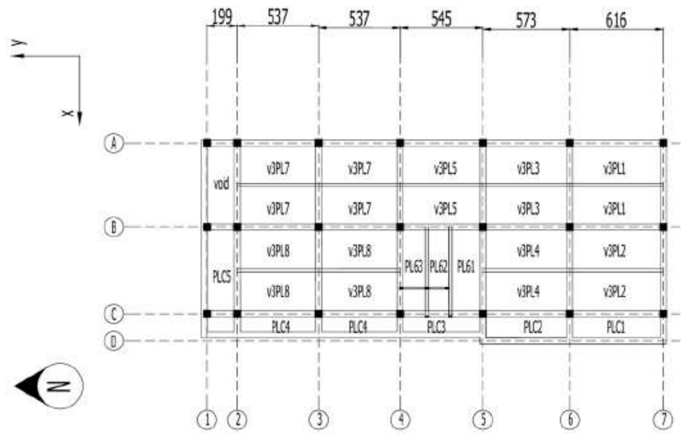

Figure 5.30 Beam Codification on Stories 4 to 7 in Configuration Model 3

4. Configuration 4
   a. First mezzanine story beam and slab code.

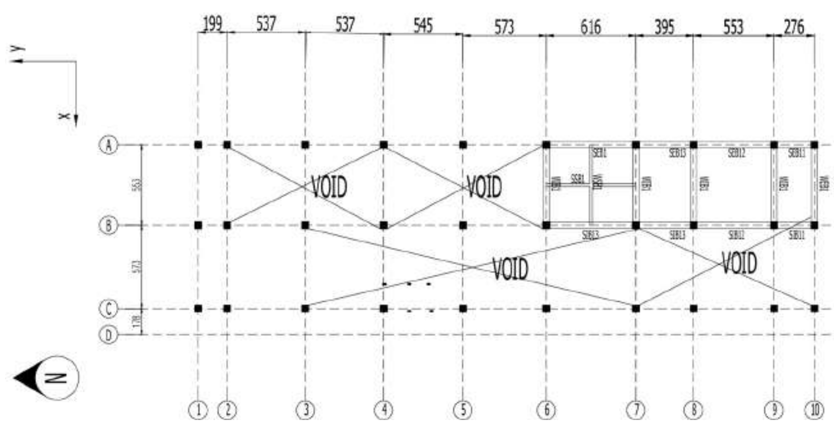

Figure 5.31 Beam Codification on Story 1.5 in Configuration Model 4

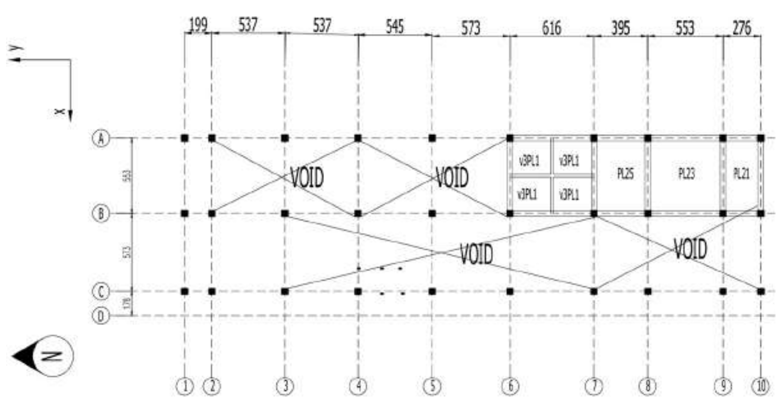

Figure 5.32 Slab Codification on Story 1.5 in Configuration Model 4

b. Second story beam and slab code.

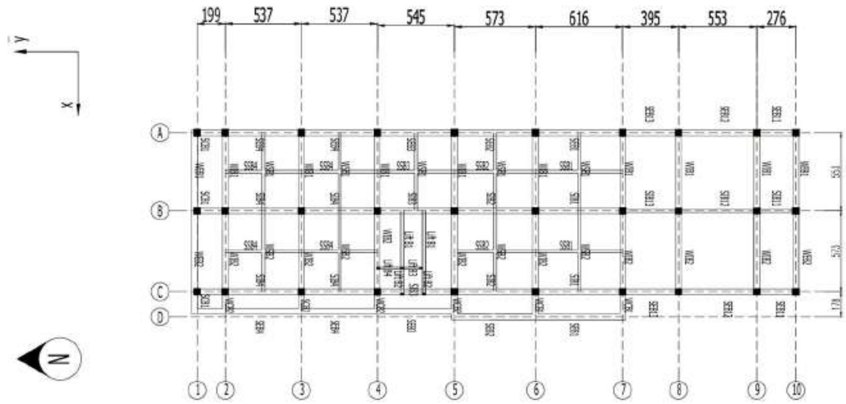

Figure 5.33 Beam Codification on Story 2 in Configuration Model 4

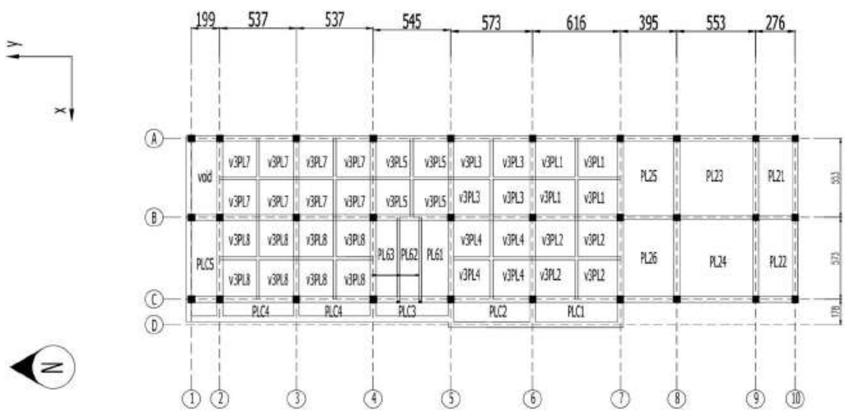

Figure 5.34 Slab Codification on Story 2 in Configuration Model 4

c. Third story beam and slab code.

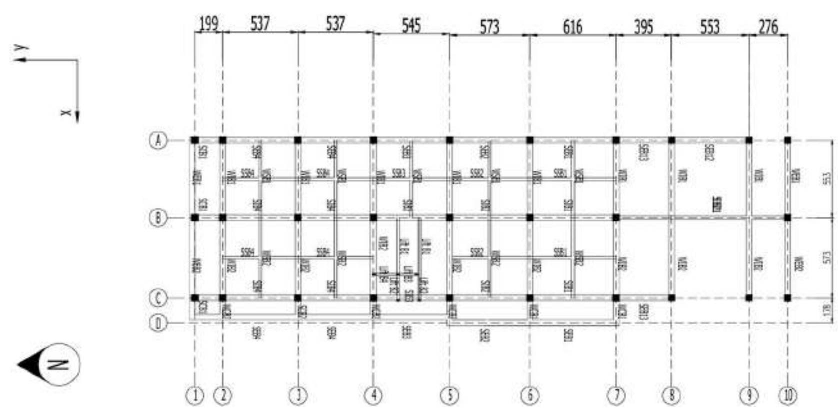

Figure 5.35 Beam Codification on Story 3 in Configuration Model 4

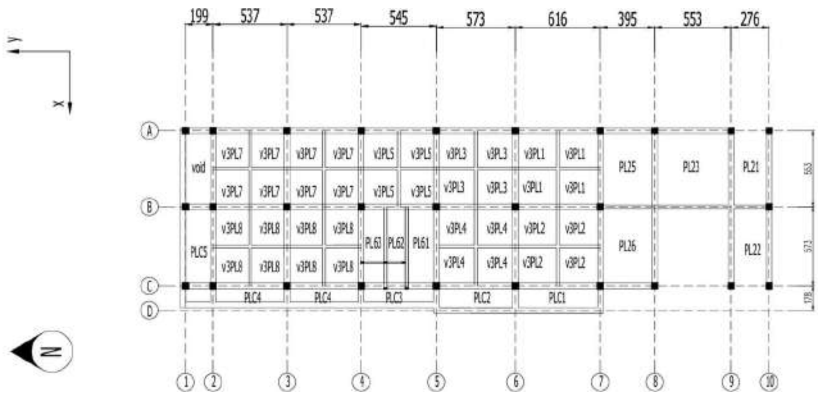

Figure 5.36 Slab Codification on Story 3 in Configuration Model 4

d. Fourth to seventh story beam and slab code.

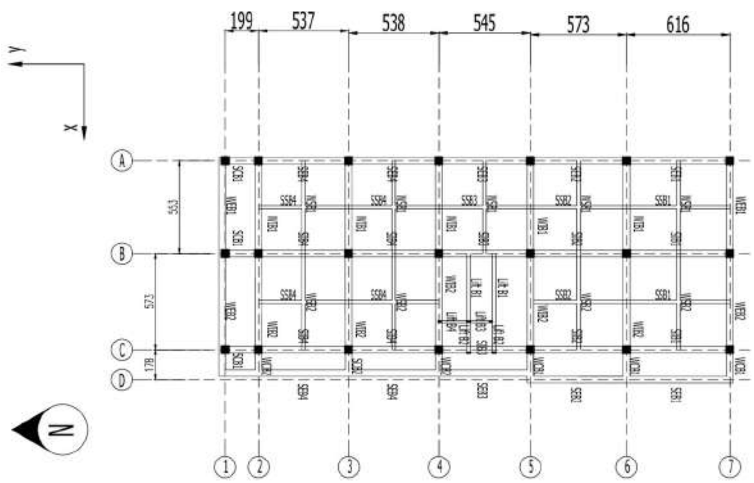

Figure 5.37 Beam Codification on Stories 4 to 7 in Configuration Model 4

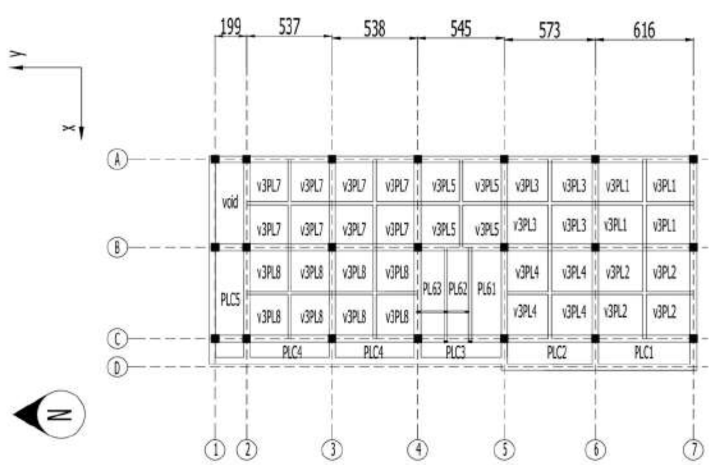

Figure 5.38 Slab Codification on Stories 4 to 7 in Configuration Model

5. Configuration 5

   a. First mezzanine story beam and slab code.

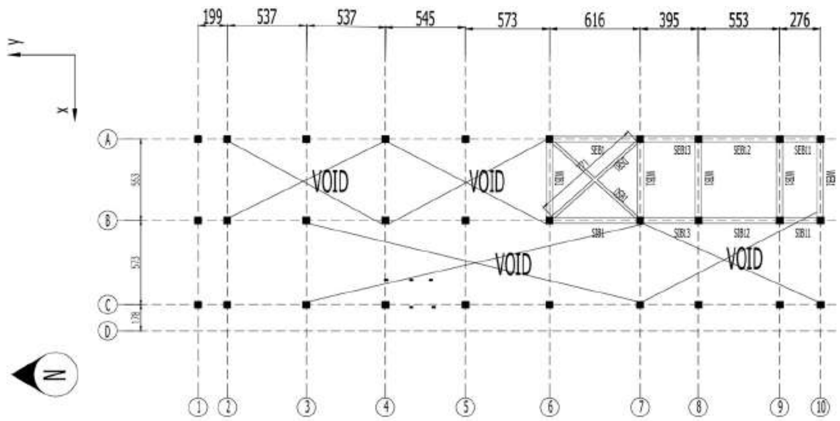

Figure 5.39 Beam Codification on Story 1.5 in Configuration Model 5

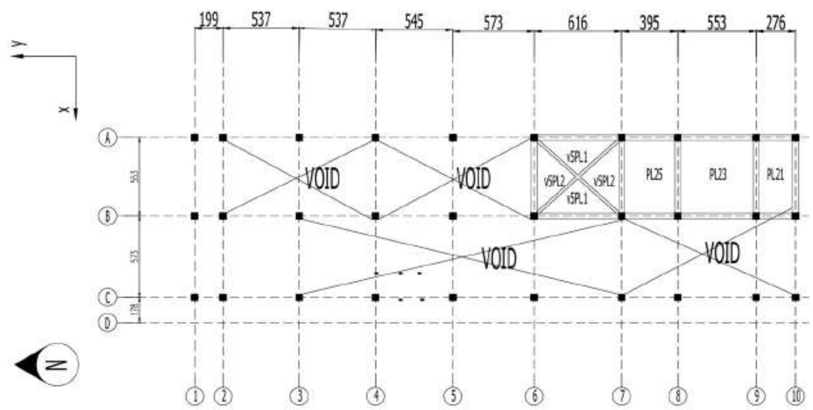

Figure 5.40 Slab Codification on Story 1.5 in Configuration Model 5

   b. Second story beam and slab code.

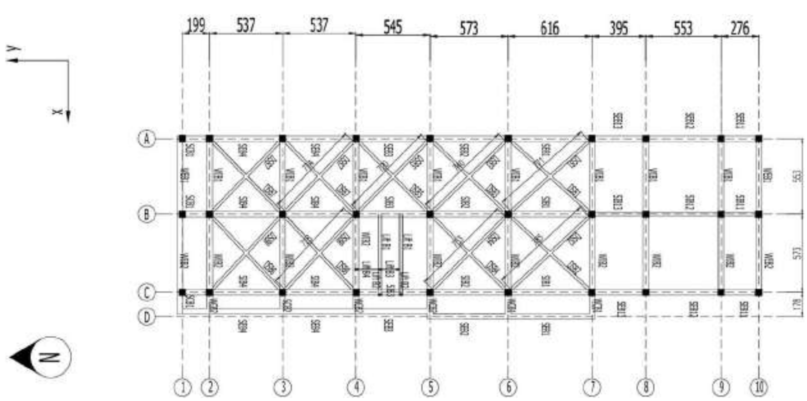

Figure 5.41 Beam Codification on Story 1.5 in Configuration Model 5

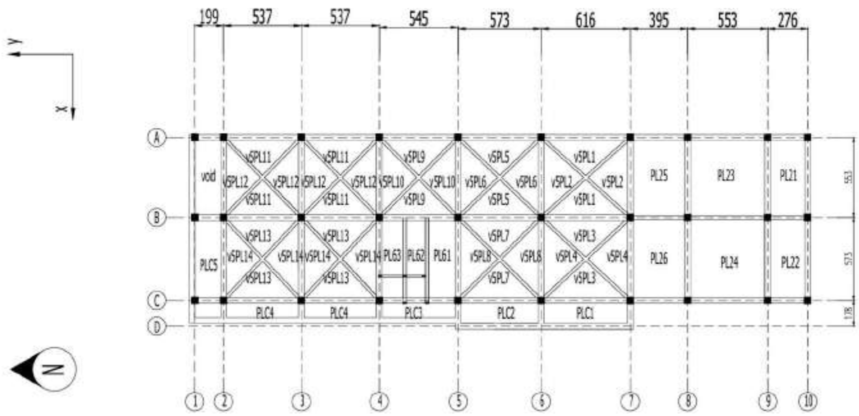

Figure 5.42 Slab Codification on Story 2 in Configuration Model 5

c. Third story beam and slab code.

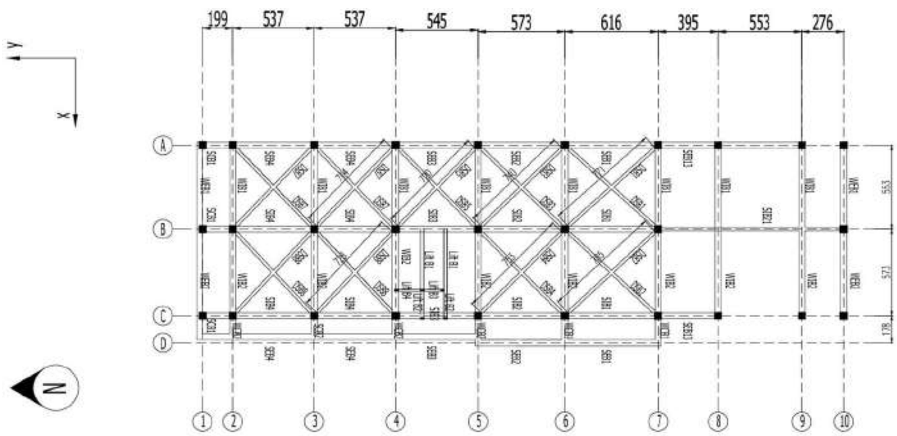

Figure 5.43 Beam Codification on Story 3 in Configuration Model 5

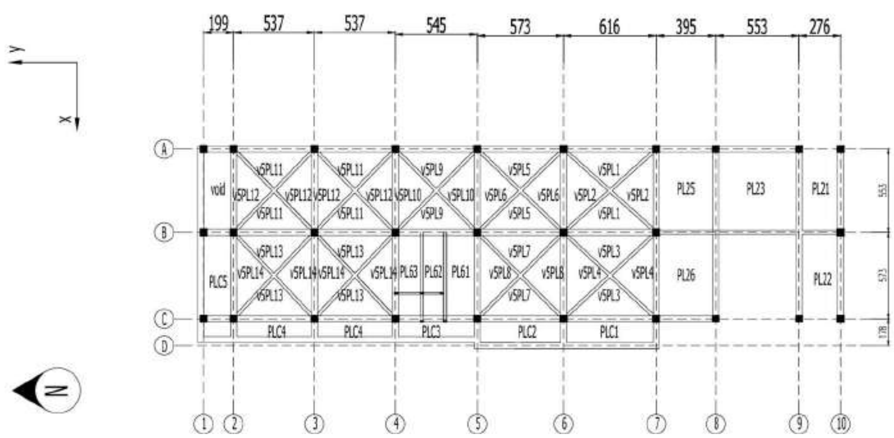

Figure 5.44 Slab Codification on Story 3 in Configuration Model 5

d. Fourth to seventh story beam and slab code.

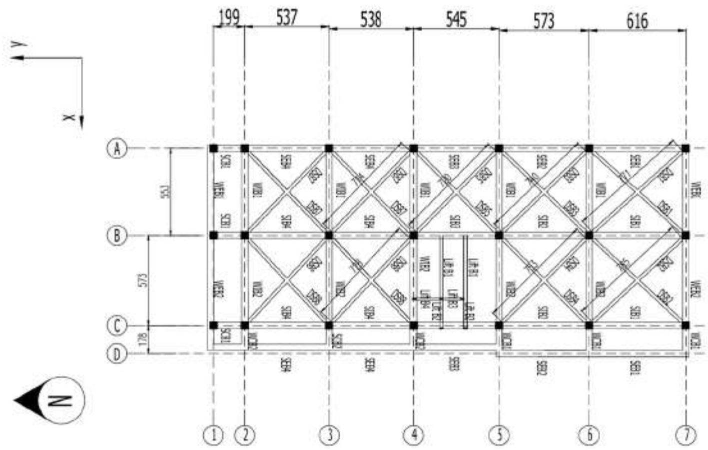

Figure 5.45 Beam Codification on Stories 4 to 7 in Configuration Model 5

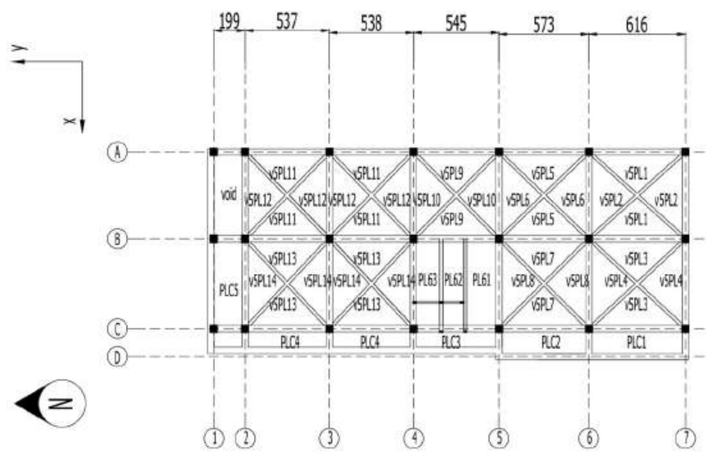

Figure 5.46 Slab Codification on Stories 4 to 7 in Configuration Model 5

5.3 Structure Element Preliminary

The structure element preliminary calculated in accordance with the code (BSN, 2019b). The structure model element apply the minimum dimension requirement as a purpose to achieve significant influence from secondary beam configurations.

5.3.1 Beam preliminary

The beams preliminary is calculated with equation in Table 3.13 for exterior beams that supporting the window and Table 3.12 for interior beams that supporting insensitive non-structural element.

The calculation of beam code SEB2 with the span length 5730 mm as follows.

$$h_b = \frac{l}{11}$$

$$h_b = \frac{5730}{11}$$

$$h_b = 520.909 \approx 521 \text{ mm}$$

The recapitulation of the rest beams can be seen in the Table 5.1 for main beam and Table 5.2 for secondary beam.

Table 5.1 Main Beam Dimension

Code	h_b (mm)	B (mm)
WEB1	395	250
WEB2	410	250
WEB1	395	250
WIB1	461	250
WIB2	478	250
SEB11	251	250
SEB12	503	252
SEB13	360	250
SEB1	560	280
SEB2	521	261
SEB3	496	250
SEB4	489	250
SCB1	398	250
SIB11	250	250
SIB12	461	250
SIB13	330	250
SIB1	514	257
SIB2	478	250
SIB3	455	250
SIB4	448	250

Table 5.2 Secondary Beam Dimension

Code	h_b (mm)	B (mm)
Lift B	254	250
WSB1	346	250
WSB2	359	250
SSB1	385	250
SSB2	359	250
SSB3	341	250
SSB4	336	250
DSB1	482	250
DSB2	491	250
DSB3	463	250
DSB4	471	250
DSB5	450	250
DSB6	447	250
DSB7	455	250

5.3.2 Slab Preliminary

Slab preliminary to obtain minimum thickness of the slab. Minimum thickness of the slab has a purpose to avoid excessive deflection that cause crack. The example calculation of the slab code V1PL1 preliminary with assumption thickness 120 mm is as follows.

$ly \quad = 6160 \text{ mm}$

$lx \quad = 5530 \text{ mm}$

$ib_{\text{WEB1}} \quad = \frac{1}{12} bh^3$

WEB1 $hb \quad = 395 \text{ mm}$

WEB1 $wb \quad = 250 \text{ mm}$

$ib_{\text{WEB1}} \quad = \frac{1}{12} \times 250 \times 395^3$

$ib_{\text{WEB1}} \quad = 1.3 \times 10^9 \text{ mm}^4$

WIB1 $hb \quad = 426 \text{ mm}$

WIB1 $wb \quad = 250 \text{ mm}$

$ib_{\text{WIB1}} \quad = \frac{1}{12} \times 250 \times 426^3$

$ib_{\text{WIB1}} \quad = 1.6 \times 10^9 \text{ mm}^4$

SEB1 $hb \quad = 560 \text{ mm}$

SEB1 $wb \quad = 280 \text{ mm}$

$ib_{\text{SEB1}} \quad = \frac{1}{12} \times 280 \times 560^3$

$ib_{\text{SEB1}} \quad = 4.1 \times \text{mm}^4$

SIB1 $hb \quad = 474 \text{ mm}$

SIB1 $wb \quad = 250 \text{ mm}$

$ib_{\text{SIB1}} \quad = \frac{1}{12} \times 250 \times 474^3$

ib_{SIB1} $= 2.2 \times$

ib_{WEB1} $= \frac{1}{12} \times 250 \times 395^3$

lny $= 5895$ mm

lnx $= 5280$ mm

i_{sly} $= 8.5 \times 10^8$ mm

i_{slx} $= 7.6 \times 10^8$

a_f $= \frac{i_b}{I_s}$

a_{f1} $= 1.689$

a_{f2} $= 2.118$

a_{f3} $= 4.287$

a_{f4} $= 2.614$

a_{fm} $= 2.812$

a_{f1} $= 1.689$

β $= 1.114$ (2-way)

h_s $= \frac{l_{ny}(0.8+\left(\frac{fy}{1400}\right)}{36+9\beta} > 90$

h_s $= \frac{5895\times(0.8+\left(\frac{420}{1400}\right)}{36+\ (1.114)} > 90$

h_s $= 140.89 > 90$

h_s $= 141$ mm

The recapitulation of the slab preliminary of another slab can be seen in Table 5.3.

Table 5.3 Recapitulation of Slab Thickness

Code	h_s (mm)
PLC1	90
PLC2	90
PLC3	90
PLC4	90
PLC5	90
PL21	105
PL22	105
PL23	130
PL24	133
PL25	120
PL26	123
PL61	90
PL62	90
PL63	90
V1PL1	141
V1PL2	143
V1PL3	133
V1PL4	134
V1PL5	129
V1PL7	129
V1PL8	133
V2PL1	112
V2PL2	115
V2PL3	109
V2PL4	112
V2PL5	104
V2PL7	102
V2PL8	102
V3PL1	116
V3PL2	120
V3PL3	116
V3PL4	111
V3PL5	107
V3PL7	106
V3PL8	107
V4PL1	90
V4PL2	90
V4PL3	90
V4PL4	90
V4PL5	90
V4PL7	90
V4PL8	90
V5PL1	116
V5PL2	129
V5PL3	120
V5PL4	129
V5PL5	116
V5PL6	120
V5PL7	120
V5PL8	120
V5PL9	116
V5PL10	114
V5PL11	120
V5PL12	112
V5PL13	116
V5PL14	112

5.3.3 Column Preliminary

Column preliminary to obtain minimum dimension of column. Minimum dimension of the column has a purpose to make column capacity can withstand building load. The calculation of the column uses tributary areas load method. Based on the column tributary areas load, the load is around 2400000. The preliminary calculation of column with 3.6 meters length is as follows.

$$Ag = \frac{N}{0.35 \times f_c{}'}$$

$$Ag = \frac{2400000}{0.35 \times 25}$$

$$Ag = 274285.71 \text{ mm}^2$$

$$b, h = \sqrt{274285.71}$$

$$b \approx 550 \text{ mm}$$

$$h \approx 550 \text{ mm}$$

While for column with 7 meters length is 850 mm × 550 mm to avoid column slenderness.

5.3.4 Building Mass

The building mass is computed each grid and each element manually to verify the result in structural analysis program. The calculations to obtain structure weight by multiplying member volume with reinforce concrete density γ_{rc}. Were, $\gamma_{rc} = 23.544 \text{ kN/m}^3$.

$$\text{SEB2 Weight} = \gamma_{rc} \times l_n \times H \times B$$

$$= 23.544 \times \left(\frac{6160 \times 560 \times 280}{10^9}\right)$$

$$= 22.741 \text{ kN}$$

The recapitulation of the rest member and other model variation is computed by Microsoft excel can be seen in Table 5.4 to Table 5.8.

Table 5.4 Variation 1 Building Mass

Element Story	Member Weight (kN)					Story weight
	Beam	Column	Slab	Wall	ADL	
7	575.150	290.403	1092.252	0.000	474.701	2432.506
6	575.150	580.807	1092.252	945.283	816.987	4010.479
5	575.150	580.807	1092.252	945.283	816.987	4010.479
4	575.150	580.807	1092.252	945.283	816.987	4010.479
3	798.893	619.690	1389.938	945.283	816.987	4570.791
2	772.904	1268.445	1489.160	1166.259	1116.463	5813.231
1.5	170.468	348.981	306.130	252.298	221.098	1298.975
1	0.000	949.129	0.000	0.000	0.000	949.129
Sum	4042.866	5219.069	7554.235	5199.690	5080.210	27096.071

Table 5.5 Variation 2 Building Mass

Element	Weight (kN)					Story weight
Story	Beam	Column	Slab	Wall	ADL	
7	679.893	290.403	910.403	0.000	474.701	2355.400
6	679.893	580.807	910.403	945.283	816.987	3933.373
5	679.893	580.807	910.403	945.283	816.987	3933.373
4	679.893	580.807	910.403	945.283	816.987	3933.373
3	903.636	619.690	1208.088	945.283	950.988	4627.685
2	877.646	1268.445	1307.311	1166.259	1116.463	5736.125
1.5	170.468	348.981	282.871	252.298	221.098	1275.717
1	0.000	949.129	0.000	0.000		949.129
sum	4671.321	5219.069	6439.882	5199.690	5214.211	26744.173

Table 5.6 Variation 3 Building Mass

Element	Weight (kN)					Story weight
Story	Beam	Column	Slab	Wall	ADL	
7	680.704	290.403	937.182	0.000	474.701	2382.990
6	680.704	580.807	937.182	945.283	816.987	3960.963
5	680.704	580.807	937.182	945.283	816.987	3960.963
4	680.704	580.807	937.182	945.283	816.987	3960.963
3	904.447	619.690	1234.868	945.283	950.988	4655.276
2	878.458	1268.445	1334.090	1166.259	1116.463	5763.715
1.5	198.386	348.981	286.080	252.298	221.098	1306.843
1	0.000	949.129	0.000	0.000		949.129
sum	4704.109	5219.069	6603.765	5199.690	5214.211	26940.844

Table 5.7 Variation 4 Building Mass

Element	Weight (kN)					Story weight
Story	Beam	Column	Slab	Wall	ADL	
7	785.447	290.403	796.701	0.000	474.701	2347.252
6	785.447	580.807	796.701	945.283	816.987	3925.224
5	785.447	580.807	796.701	945.283	816.987	3925.224
4	785.447	580.807	796.701	945.283	816.987	3925.224
3	1009.190	619.690	1094.386	945.283	950.988	4619.537
2	983.201	1268.445	1193.609	1166.259	1116.463	5727.976
1.5	170.468	348.981	265.227	252.298	221.098	1258.072
1	0.000	949.129	0.000	0.000		949.129
sum	5304.645	5219.069	5740.025	5199.690	5214.211	26677.640

Table 5.8 Variation 5 Building Mass

Element	Weight (kN)					Story weight
Story	Beam	Column	Slab	Wall	ADL	
7	937.626	290.403	984.487	0.000	474.701	2687.218
6	937.626	580.807	984.487	945.283	816.987	4265.191
5	937.626	580.807	984.487	945.283	816.987	4265.191
4	937.626	580.807	984.487	945.283	816.987	4265.191
3	1161.369	619.690	1282.173	945.283	950.988	4959.503
2	1135.380	1268.445	1381.396	1166.259	1116.463	6067.942
1.5	214.215	348.981	291.293	252.298	221.098	1327.885
1	0.000	949.129	0.000	0.000	0.000	949.129
sum	6261.469	5219.069	6892.810	5199.690	5214.211	28787.249

5.4 Seismic Design Requirements

Building in the Ngawi district, West Java, Indonesia, short period and one-second period spectral response parameters S_s and S_1 according to map at Figure 3.2 and Figure 3.1 are 0.888 g and 0.380 g, respectively. For soft soil condition with site class D.

Short period site F_a is obtained 1.145 by using the interpolation of $S_s = 0.888$ and site class D in accordance with Table 3.2. while the velocity-based site coefficient, F_v is obtained 1.920 by using the interpolation of $S_1 = 0.380$ and site class D in accordance with Table 3.3.

The hotel building adjusted maximum considered spectral response acceleration parameter are in accordance with equation (3.1) and (3.2).

$$S_{MS} = 1.145 \times 0.888$$

$$= 1.106$$

$$S_{M1} = F_v S_1$$

$$= 1.920 \times 0.380$$

$$= 0.730$$

The calculation of design spectral response acceleration parameters in accordance with equation (3.3) and (3.4).

$$S_{DS} = 2/3\ S_{MS}$$

$$= \frac{2}{3} \times 1.106$$

$$= 0.677$$

$$S_{D1} = \frac{2}{3} S_{M1}$$

$$= \frac{2}{3} \times 0.730$$

$$= 0.486$$

5.5 Building and Structure Description

The building function will be used as hotel that the seismic force resisting system is special moment resisting frame. Based on the function the building is classified as residential building. Thus, the building risk category in accordance with code (BSN, 2019a) is category II. Following the risk category the earthquake important factor I_e is 1.0.

5.5.1 Seismic Design Category

Since the S_1 value is 0.380 g the seismic design category is classified in accordance with Table 3.4 and Table 3.5. Where, S_{DS} is 0.677 g and S_{D1} is 0.486 g then the seismic design category D is obtained.

5.5.2 Structure system factors

Building structure system applies special moment resisting reinforced concrete frame with seismic design category D. Based on the Table 3.6 the factors of the structure system are:

1. Response modification coefficient, R is 8,
2. Overstrength factor, Ω is 3, and
3. Deflection magnification factor, C_d is 5.5.

5.6 Building Mass Analysis

The structure element is arranged into the Software analysis program as the open frame structure with the slab supported by beams.

5.6.1 Building mass result

Software analysis program computes every member's self-load and additional load that drawn through the base reaction. Thus, the building mass is obtained. The software analysis program's building mass is compared with manual calculate building mass to verify the structure model.

Table 5.9 Weight Control of Variation 1 Model

Parameter	Weight (kN)		Error
	SAP2000	Manual	
DEAD	16851.089	16816.170	0.207%
ADL	10211.964	10279.900	0.665%
Building Mass	27208.271	27096.071	0.412%
Control			**Valid**

Based on Table 5.9 the model in the software analysis is valid since the error is small. The weight control is repeated for other model variations to verify the other models. The weight control for other models can be seen in Table 5.10 to Table 5.13.

Table 5.10 Weight Control of Variation 2 Model

Parameter	Weight (kN)		Error
	Software	Excel	
DEAD	16377.412	16330.272	0.288%
ADL	10357.182	10413.901	0.548%
Building mass	26734.594	26744.173	0.036%
Control			**Valid**

Table 5.11 Weight Control of Variation 3 Model

Parameter	Weight (kN)		Error
	Software	Excel	
DEAD	16548.861	16526.942	0.132%
ADL	10357.182	10413.901	0.548%
Building mass	26906.043	26940.844	0.129%
Control			**Valid**

Table 5.12 Weight Control of Variation 4 Model

Parameter	Weight (kN)		Error
	Software	Excel	
DEAD	16323.270	16263.739	0.365%
ADL	10357.182	10413.901	0.548%
Building mass	26680.452	26677.640	0.011%
Control			**Valid**

Table 5.13 Weight Control of Variation 5 Model

Parameter	Weight (kN)		Error
	Software	Excel	
DEAD	18522.347	18373.348	0.804%
ADL	10357.182	10413.901	0.548%
Building mass	28879.529	28787.249	0.320%
Control			**Valid**

The building weight control for each variation is near to 0 percent. Hence, the analysis procedure can be continued.

5.6.2 Building mass discussion

Building mass of each structure have the purpose for controlling the analysis and estimating structural material and work cost. The comparison of model variations can be seen in Figure 5.47.

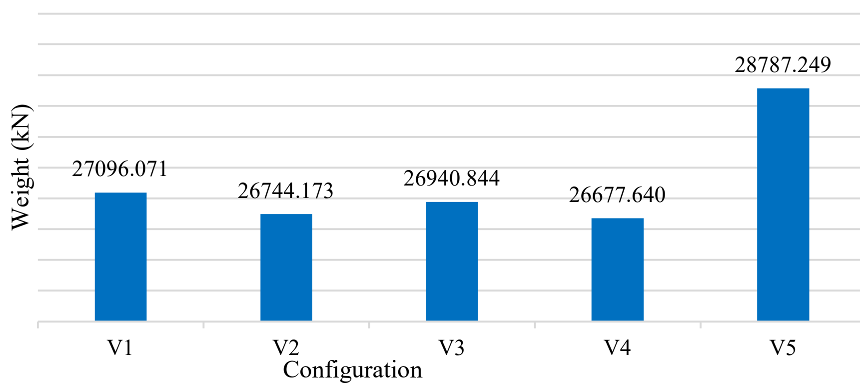

Figure 5.47 Building Mass Comparison

Based on the Figure 5.47 the model variation with the least mass is model variation 4 because secondary beam configuration, the slab thickness is allowed to apply minimum thickness. While model variation with the heaviest mass is model variation 5 because secondary beam configuration and the slab have long span. Theoretically, if structure stiffness is assumed same for each model. The structure with least mass will have least deflection and least structure period.

The most economic structure based on Figure 5.47 is model variation 4 since has least mass and thinnest slab. Model variation 4 has 1.5% less mass than structure without secondary beam (Model variation 1). While the heaviest variation is model variation 5 which has 6.2% more mass than structure without secondary beam (model variation 1).

5.7 Building Diaphragm

The beam, column, and slab will be cast into monolith structure. Consequently, Define the diaphragm for each story in software analysis program is necessary. Define the diaphragm type by lateral equivalent load tributary, *S* to unidirectional frame against lateral load D_e ratio.

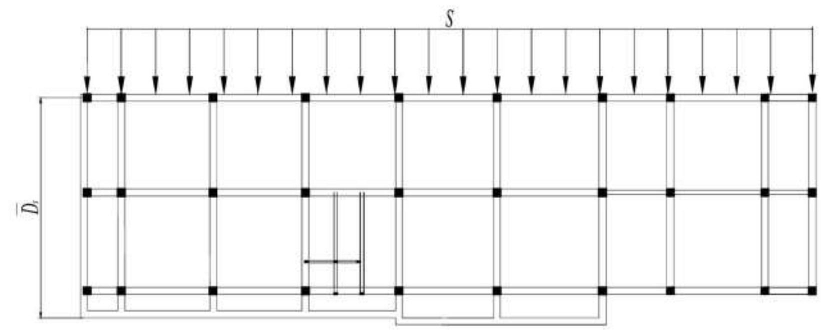

Figure 5.48 Tributary Lateral Force in Strong axis

Based on the Figure 5.48 the ratio is as follows.

$$\frac{S}{D_e} = \frac{42.32}{12.84}$$

$$\frac{S}{D_e} = 3.300$$

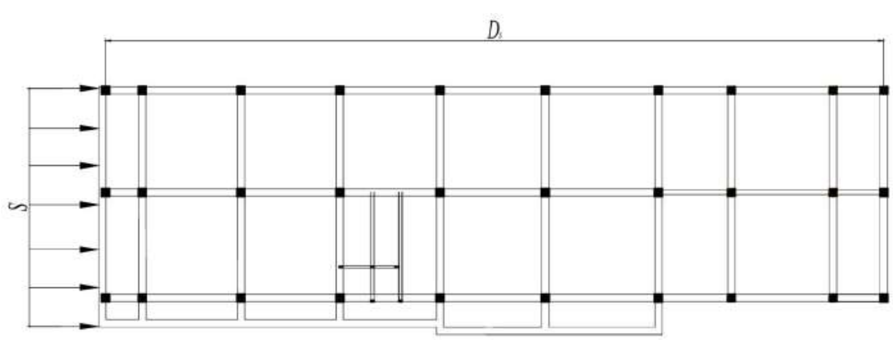

Figure 5.49 Tributary Lateral Force in Weak Axis

Based on the Figure 5.49 the ratio is as follows.

$$\frac{S}{D_e} = \frac{12.83}{42.32}$$

$$\frac{S}{D_e} = 0.303$$

The value of S/D_e ratio in strong and weak axis direction has significant differences. Consequently, the value taken is the worse condition, i.e. 3.300. Conservatively, the diaphragm is defined as semi rigid diaphragm. The diaphragm defined in software analysis through define the join constraint can be seen in Figure 5.50

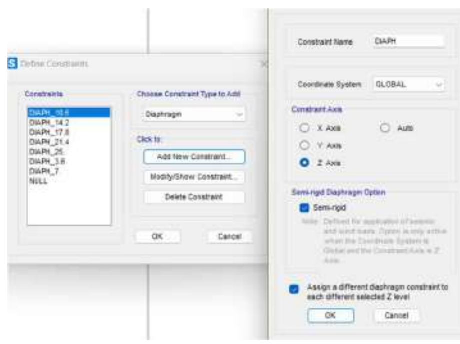

Figure 5.50 Defines the Diaphragm Model

5.7.1 Element section stiffness

The joint in the structures should be in the critical condition. Therefore, every joint in the model is set to have plastic condition by applying the section stiffness reduction factor through property modifier in software analysis program.

The stiffness reduction factor in accordance with code are 0.35 times beam inertia and 0.70 times column inertia (BSN, 2019b). The input to the software analysis can be seen in Figure 5.51.

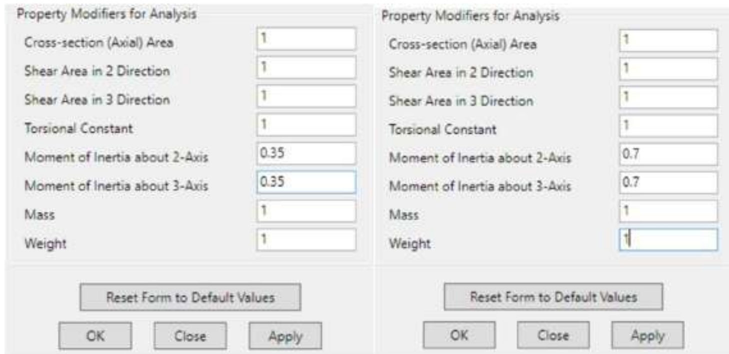

Figure 5.51 Beam and Column Properties Modifier

5.8 Structure period

The structure period for the structure in plastic joints condition is obtained by running the analysis with the modal.

5.8.1 Structure period Result

The period taken is the maximum period each model variations, the value can be seen in recapitulation Table 5.14.

Table 5.14 Structure Period Recapitulation in Plastic Joints Condition

T_c (s)				
V1	**V2**	**V3**	**V4**	**V5**
1.760	1.914	1.865	1.945	1.761

Comparison of each model variation's structure period that obtained by running the analysis with the modal can be seen in Figure 5.52

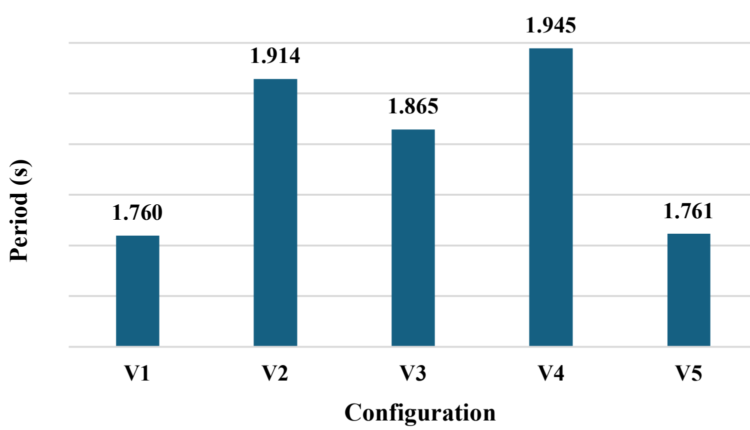

Figure 5.52 Structure Period Comparison

Based on the Figure 5.52 the structure with secondary beam with least period is model variation 5, it is contradicted based on the discussion section 5.6.2. Consequently, the stiffness of the structure cannot be assumed same for other model variations. In other words, the secondary beam configuration has influence to structure response.

5.8.2 Structure period discussion

Measure the stiffness of structure can be simplified with the help of the structural analysis program based on Figure 5.52. Since the period of the structure can be obtained from the program, the structure stiffness can be calculated in accordance with theory single degree of freedom structure (Paulay & Priestley, 1992) equation (3.14).

$$K = \left(2\Pi \times \frac{\sqrt{w}}{T\times\sqrt{g}}\right)^2$$

$$K = \left(2\Pi \times \frac{\sqrt{27208.271}}{1.760\times\sqrt{9.81}}\right)^2$$

$$K = 3590026.580 \text{ kN/m}$$

The structure stiffness of the rest variations can be seen in the Table 5.15

Table 5.15 Recapitulation of Structure Stiffness

Model Stiffness (kN/m)				
V1	**V2**	**V3**	**V4**	**V5**
3590026.580	2993643.262	3179036.846	2893903.202	3806325.336

Based on Table 5.15 each model variation has different stiffness. The structure model 5 is the structure with stiffest stiffness, in accordance with the (Paulay & Priestley, 1992) the structure model 5 inter-story deflection will lower than another model. The secondary beam configuration of model 5 has most influence the stiffness for the whole structure.

5.9 Equivalent Lateral Force (ELF)

Perform ELF analysis is necessary in structural analysis. ELF analysis is used for preliminary design and evaluating the irregularities.

5.9.1 Equivalent lateral force result

The calculation of the ELF is depended by structure mass and period in plastic joint condition (BSN, 2019a) for structure variation 1 can be seen as follows.

$$C_{s1} = \frac{0.677}{\left(\frac{8}{1}\right)}$$

$$= 0.085$$

$$Tc = 1.76614 \text{ sec}$$

$$C_{s2} = \frac{0.486}{\left(1.76614 \times \left(\frac{8}{1}\right)\right)}$$

$$= 0.034$$

$$C_{smin} = 0.044 \times 0.677 \times 1$$

$$= 0.030$$

The seismic response coefficient used is C_{s2} since the ELF is used to evaluate the structure irregularities.

$$W = 27096.071 \text{kN}$$

$$V = 0.034 \times 27096.071$$

$$= 932.732 \text{ kN}$$

$$k = 1 - \frac{(1.76614 - 0.5)}{(2.5 - 0.5)} \times (1 - 2)$$

$$= 1.633$$

The lateral force for each story is computed in accordance with equation (3.15) and (3.16). Based on the equation, the result is obtained in the Table 5.16. to Table 5.20.

Table 5.16 ELF of Structure Variation 1

Story	Wt **(kN)**	ht **(m)**	$Wt.ht^k$	Cvx	Fx **(kN)**
7	2432.51	25	466646.343	0.214	199.973
6	4010.48	21.4	596836.117	0.274	255.763
5	4010.48	17.8	441793.853	0.203	189.323
4	4010.48	14.2	305466.650	0.140	130.902
3	4570.79	10.6	215967.097	0.099	92.549
2	5813.23	7	139483.183	0.064	59.773
1.5	1298.98	3.6	10521.601	0.005	4.509
1	949.129	0	0.000	0.000	0.000
Sum			2176714.800	1	

Table 5.17 ELF of Structure Variation 2

Story	Wt **(kN)**	ht **(m)**	$Wt.ht^k$	Cvx	Fx **(kN)**
7	2355.400	25	573595.090	0.217	184.546
6	3933.373	21.4	734558.176	0.278	236.333
5	3933.373	17.8	536367.075	0.203	172.568
4	3933.373	14.2	364697.944	0.138	117.336
3	4627.685	10.6	260465.120	0.099	83.801
2	5736.125	7	158985.099	0.060	51.151
1.5	1275.717	3.6	11362.267	0.004	3.656
1	949.129	0	0.000	0.000	0.000
Sum			2640030.772	1	
K	1.707				
V	849.390				

Table 5.18 ELF of Structure Variation 3

Story	Wt **(kN)**	ht **(m)**	$Wt.ht^k$	Cvx	Fx **(kN)**
7	2382.990	25	535581.702	0.216	189.882
6	3960.963	21.4	685341.998	0.277	242.977
5	3960.963	17.8	502732.249	0.203	178.236
4	3960.963	14.2	343758.495	0.139	121.874
3	4655.276	10.6	247047.266	0.100	87.587
2	5763.715	7	152187.891	0.061	53.956
1.5	1306.843	3.6	11273.815	0.005	3.997
1	949.129	0	0.000	0.000	0.000
Sum			2477923.416	1	
K	1.682				
V	878.509				

Table 5.19 ELF of Structure Variation 4

Story	Wt (kN)	ht (m)	$Wt.ht^k$	Cvx	Fx (kN)
7	2347.252	25	600164.292	0.218	182.006
6	3925.224	21.4	767843.528	0.279	232.856
5	3925.224	17.8	559109.995	0.203	169.556
4	3925.224	14.2	378863.183	0.138	114.894
3	4619.537	10.6	269470.322	0.098	81.720
2	5727.976	7	163507.175	0.059	49.585
1.5	1258.072	3.6	11424.592	0.004	3.465
1	949.129	0	0.000	0.000	0.000
Sum			2750383.087	1	
K	1.722				
V	834.081				

Table 5.20 ELF of Structure Variation 5

Story	Wt (kN)	ht (m)	$Wt.ht^k$	Cvx	Fx (kN)
7	2687.218	25	511591.833	0.221	219.129
6	4265.191	21.4	630150.341	0.272	269.911
5	4265.191	17.8	466657.589	0.201	199.882
4	4265.191	14.2	322830.857	0.139	138.277
3	4959.503	10.6	233026.031	0.100	99.811
2	6067.942	7	144924.839	0.062	62.075
1.5	1327.885	3.6	10723.167	0.005	4.593
1	949.129	0	0.000	0.000	0.000
Sum			2319904.656	1	
K	1.631				
V	993.678				

5.9.2 Equivalent lateral force discussion

The base shear and equivalent lateral force analysis have relation to structure mass and structure period. structure with more mass causing higher base shear and/or structure with longer structure period causing higher base shear also. Hence, the equivalent lateral force will increase affected by base shear and vice versa. The Base shear of each model variation can be seen Figure 5.53.

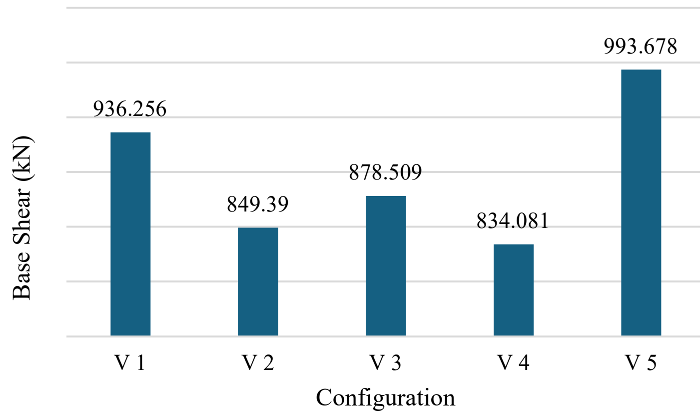

Figure 5.53 Base Shear Result Comparison

Based on Figure 5.53 structure model variation 4 is the structure with the lowest base shear. While structure model variation 5 is the structure with the highest base shear. The result of base shear comparison based on the section 5.6 and 5.8 have corresponding relationships.

Equivalent lateral force analysis with base shear resulting the equivalent lateral force comparison for each model variations. The result can be seen in Figure 5.54

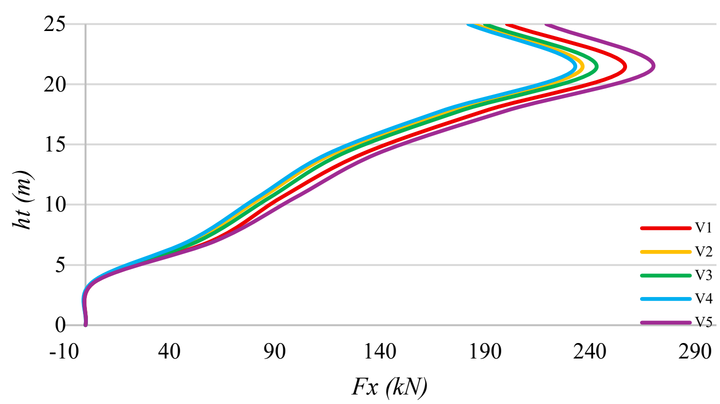

Figure 5.54 Equivalent Lateral Force Comparison

Based on the equivalent lateral force analysis result, the structure model variation 5 is expected to have more lateral force in its each story.

5.10 Response Spectrum

The design response spectrum for the structure is as follows.

The computation for response spectrum in period lower than T_0, the S_a in accordance with equation (3.5) as follows:

$$T_0 = 0.2 \times \frac{0.677}{0.486}$$

$$= 0.144 \text{ sec}$$

$$T = 0.100 \text{ sec}$$

$$Sa = 0.677 \times \left(0.4 + 0.6 \times \left(\frac{0.100}{0.144}\right)\right)$$

$$= 0.554 \text{ g,}$$

the computation for response spectrum in period greater than or equal to T_0 and less than or equal to T_s, the S_a is equal to S_{DS}

$$Ts = \frac{0.677}{0.486}$$

$$= 0.718 \text{ sec}$$

$$S_a = S_{DS}$$

$$= 0.677 \text{ g,}$$

the computation for response spectrum in period greater than T_s, the S_a in accordance with equation (3.6) as follows:

$$S_a = \frac{0.486}{0.800}$$

$$= 0.608 \text{ g,}$$

the design response spectrum based on the computation can be seen in Figure 5.55

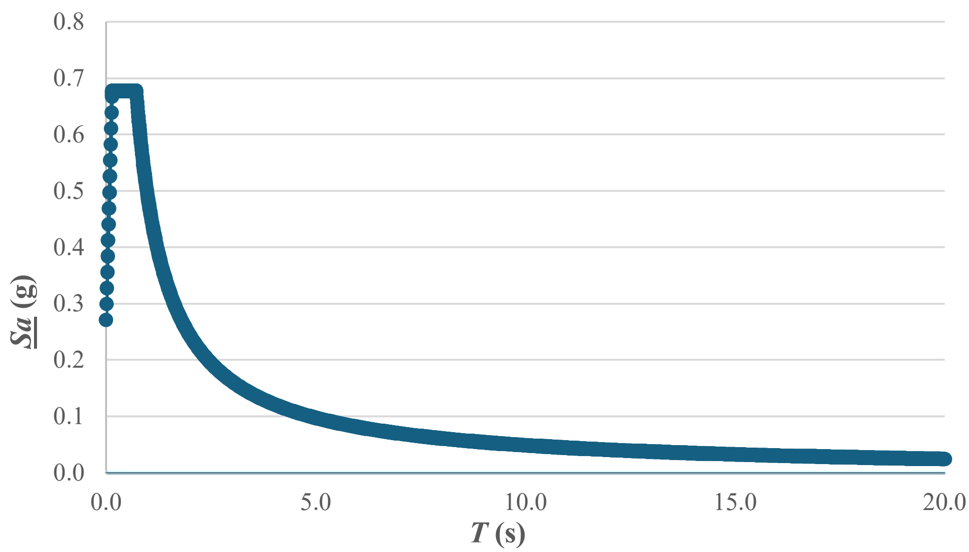

Figure 5.55 Response Spectrum Design

5.11 Story stiffness

Story stiffness is obtained by analyse the structure deformation of structural analysis result with 1 unit load forces applied in weak axis direction and strong axis direction. Deformation is taken in column joint that closest to structure centre mass.

5.11.1 Story stiffness result

The calculation of the story stiffness in accordance with Figure 3.5 equation. The calculation of 4th story stiffness in weak axis direction load is as follows:

$$K_4 = \frac{(F_7+F_6+F_5+F_4)}{\Delta_4-\Delta_3}$$

$$K_4 = \frac{1+1+1+1}{0.109-0.077}$$

$$K_4 = 147.059 \text{ kN/mm}$$

The remaining level of story stiffnesses are presented in the table. Also, the story stiffness of each model variation can be seen in Table 5.21 to Table 5.30.

Table 5.21 Story Stiffness in Weak Axis Direction Model 1

Story	Story Height	∑F	U1	Δ_i	K_i	SNI	Check	SNI	Check
	m	Vx (kN)	mm	mm	kN/mm	70%K_{i+1}		80% (AVG.K_{i+3})	
7	25	1	0.166	0.011	90.909				
6	21.4	2	0.155	0.019	105.263	63.636	Regular		
5	17.8	3	0.136	0.027	111.111	73.684	Regular		
4	14.2	4	0.109	0.032	125.000	77.778	Regular	81.942	Regular
3	10.6	5	0.077	0.034	147.059	87.500	Regular	91.033	Regular
2	7	6	0.043	0.028	214.286	102.941	Regular	102.179	Regular
1.5	3.6	7	0.015	0.015	466.667	150.000	Regular	129.692	Regular

Table 5.22 Story Stiffness in Strong Axis Direction Model 1

Story	Story Height	∑F	U1	Δ_i	K_i	SNI	Check	SNI	Check
	m	Vx (kN)	mm	mm	kN/mm	70%K_{i+1}		80% (AVG.K_{i+3})	
7	25	1	0.260	0.019	52.632				
6	21.4	2	0.241	0.029	68.966	36.842	Regular		
5	17.8	3	0.212	0.041	73.171	48.276	Regular		
4	14.2	4	0.171	0.050	80.000	51.220	Regular	51.938	Regular
3	10.6	5	0.121	0.052	96.154	56.000	Regular	59.236	Regular
2	7	6	0.069	0.044	136.364	67.308	Regular	66.487	Regular
1.5	3.6	7	0.025	0.025	280.000	95.455	Regular	83.338	Regular

Table 5.23 Story Stiffness in Weak Axis Direction Model 2

Story	Story Height	∑F	U1	Δ_i	K_i	SNI	Check	SNI	Check
	m	Vx (kN)	mm	mm	kN/mm	70%K_{i+1}		80% (AVG.K_{i+3})	
7	25	1	0.326	0.027	37.037				
6	21.4	2	0.299	0.041	48.780	25.926	Regular		
5	17.8	3	0.258	0.054	55.556	34.146	Regular		
4	14.2	4	0.204	0.064	62.500	38.889	Regular	37.699	Regular
3	10.6	5	0.14	0.063	79.365	43.750	Regular	44.490	Regular
2	7	6	0.077	0.050	120.000	55.556	Regular	52.646	Regular
1.5	3.6	7	0.027	0.027	259.259	84.000	Regular	69.831	Regular

Table 5.24 Story Stiffness in Strong Axis Direction Model 2

Story	Story Height	∑F	U1	Δ_i	K_i	SNI	Check	SNI	Check
	m	Vx (kN)	mm	mm	kN/mm	70%K_{i+1}		80% (AVG.K_{i+3})	
7	25	1	0.184	0.014	71.429				
6	21.4	2	0.170	0.021	95.238	50.000	Regular		
5	17.8	3	0.149	0.031	96.774	66.667	Regular		
4	14.2	4	0.118	0.036	111.111	67.742	Regular	70.251	Regular
3	10.6	5	0.082	0.037	135.135	77.778	Regular	80.833	Regular
2	7	6	0.045	0.029	206.897	94.595	Regular	91.472	Regular
1.5	3.6	7	0.016	0.016	437.500	144.828	Regular	120.838	Regular

Table 5.25 Story Stiffness in Weak Axis Direction Model 3

Story	Story Height	∑F	U1	Δ_i	K_i	SNI	Check	SNI	Check
	m	Vx (kN)	mm	mm	kN/mm	70%K_i+1		80% (AVG.K_i+3)	
7	25	1	0.302	0.024	41.667				
6	21.4	2	0.278	0.036	55.556	29.167	Regular		
5	17.8	3	0.242	0.050	60.000	38.889	Regular		
4	14.2	4	0.192	0.058	68.966	42.000	Regular	41.926	Regular
3	10.6	5	0.134	0.059	84.746	48.276	Regular	49.206	Regular
2	7	6	0.075	0.048	125.000	59.322	Regular	56.990	Regular
1.5	3.6	7	0.027	0.027	259.259	87.500	Regular	74.323	Regular

Table 5.26 Story Stiffness in Strong Axis Direction Model 3

Story	Story Height	∑F	U1	Δ_i	K_i	SNI	Check	SNI	Check
	m	Vx (kN)	mm	mm	kN/mm	70%K_i+1		80% (AVG.K_i+3)	
7	25	1	0.190	0.014	71.429				
6	21.4	2	0.176	0.023	86.957	50.000	Regular		
5	17.8	3	0.153	0.032	93.750	60.870	Regular		
4	14.2	4	0.121	0.037	108.108	65.625	Regular	67.236	Regular
3	10.6	5	0.084	0.038	131.579	75.676	Regular	77.017	Regular
2	7	6	0.046	0.030	200.000	92.105	Regular	88.917	Regular
1.5	3.6	7	0.016	0.016	437.500	140.000	Regular	117.250	Regular

Table 5.27 Story Stiffness in Weak Axis Direction Model 4

Story	Story Height	∑F	U1	Δ_i	K_i	SNI	Check	SNI	Check
	m	Vx (kN)	mm	mm	kN/mm	70%K_i+1		80% (AVG.K_i+3)	
7	25	1	0.338	0.029	34.483				
6	21.4	2	0.309	0.043	46.512	24.138	Regular		
5	17.8	3	0.266	0.056	53.571	32.558	Regular		
4	14.2	4	0.21	0.066	60.606	37.500	Regular	35.884	Regular
3	10.6	5	0.144	0.065	76.923	42.424	Regular	42.850	Regular
2	7	6	0.079	0.051	117.647	53.846	Regular	50.960	Regular
1.5	3.6	7	0.028	0.028	250.000	82.353	Regular	68.047	Regular

Table 5.28 Story Stiffness in Strong Axis Direction Model 4

Story	Story Height	∑F	U1	Δ_i	K_i	SNI	Check	SNI	Check
	m	Vx (kN)	mm	mm	kN/mm	70%K_i+1		80% (AVG.K_i+3)	
7	25	1	0.194	0.015	66.667				
6	21.4	2	0.179	0.023	86.957	46.667	Regular		
5	17.8	3	0.156	0.033	90.909	60.870	Regular		
4	14.2	4	0.123	0.038	105.263	63.636	Regular	65.209	Regular
3	10.6	5	0.085	0.039	128.205	73.684	Regular	75.501	Regular
2	7	6	0.046	0.030	200.000	89.744	Regular	86.501	Regular
1.5	3.6	7	0.016	0.016	437.500	140.000	Regular	115.592	Regular

Table 5.29 Story Stiffness in Weak Axis Direction Model 5

Story	Story Height	∑F	U1	Δ_i	K_i	SNI	Check	SNI	Check
	m	Vx (kN)	mm	mm	kN/mm	70%K_i+1		80% (AVG.K_i+3)	
7	25	1	0.243	0.017	58.824				
6	21.4	2	0.226	0.027	74.074	41.176	Regular		
5	17.8	3	0.199	0.038	78.947	51.852	Regular		
4	14.2	4	0.161	0.045	88.889	55.263	Regular	56.492	Regular
3	10.6	5	0.116	0.050	100.000	62.222	Regular	64.509	Regular
2	7	6	0.066	0.041	146.341	70.000	Regular	71.423	Regular
1.5	3.6	7	0.025	0.025	280.000	102.439	Regular	89.395	Regular

Table 5.30 Story Stiffness in Strong Axis Direction Model 5

Story	Story Height	∑F	U1	Δ_i	K_i	SNI	Check	SNI	Check
	m	Vx (kN)	mm	mm	kN/mm	70%K_i+1		80% (AVG.K_i+3)	
7	25	1	0.159	0.011	90.909				
6	21.4	2	0.148	0.018	111.111	63.636	Regular		
5	17.8	3	0.130	0.025	120.000	77.778	Regular		
4	14.2	4	0.105	0.030	133.333	84.000	Regular	85.872	Regular
3	10.6	5	0.075	0.033	151.515	93.333	Regular	97.185	Regular
2	7	6	0.042	0.027	222.222	106.061	Regular	107.960	Regular
1.5	3.6	7	0.015	0.015	466.667	155.556	Regular	135.219	Regular

Based on the Table 5.21 to Table 5.30 the stiffness irregularity can be assessed. According to Table 3.10, when the story lateral stiffness is less than 70% of the story above or story lateral stiffness is less than 80% of average of 3 story above it.

5.11.2 Story stiffness discussion

The story stiffness analysis indicates the inter story stiffness ratio. The further value from standard, 70% (BSN, 2019a), the structure will have stiffer inter story stiffness ratio. The comparison result of story stiffness ratio in weak axis can be seen in Figure 5.56 and Figure 5.57.

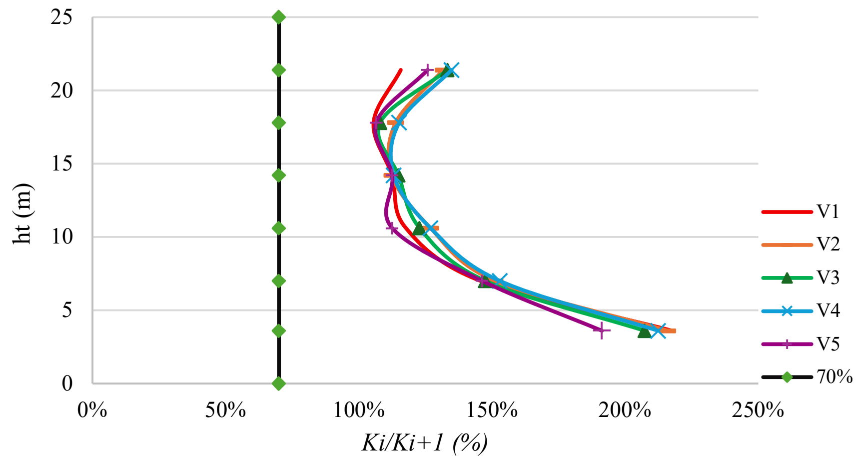

Figure 5.56 Inter Story Stiffness Ratio in Weak Axis Comparison

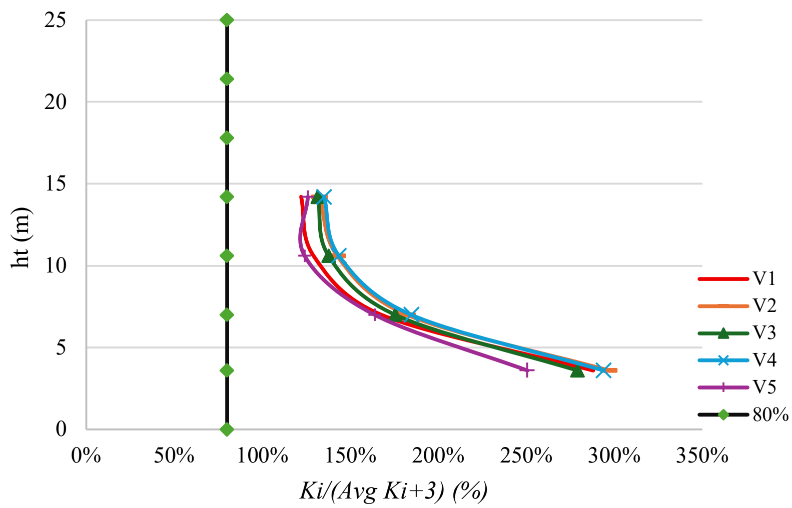

Figure 5.57 Stiffness Ratio of Three Story Above in Weak Axis Comparison

Based on Figure 5.56 and Figure 5.57 the secondary beam configuration that have most influence is model variations 2. The structure model variation 2 is the structure model with the furthest value from the standard code, 70%. Furthermore, the comparison result of story stiffness ratio in strong axis can be seen in Figure 5.58 and Figure 5.59.

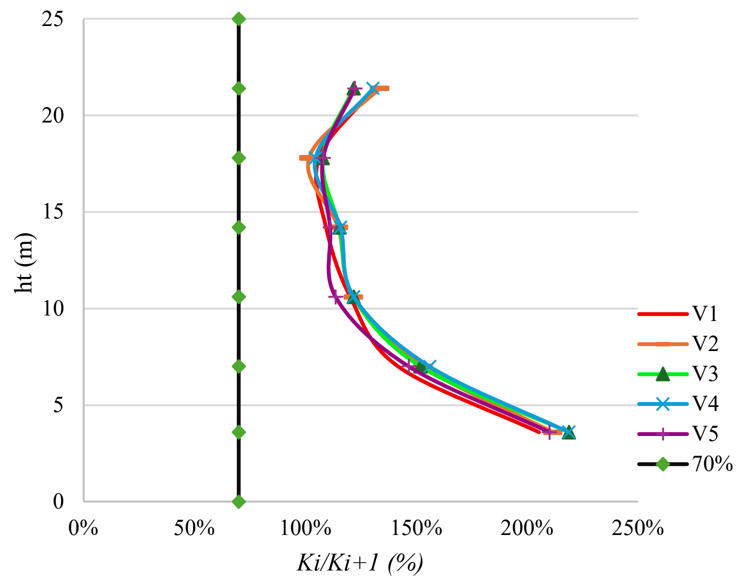

Figure 5.58 Inter Story Stiffness Ratio in Strong Axis Comparison

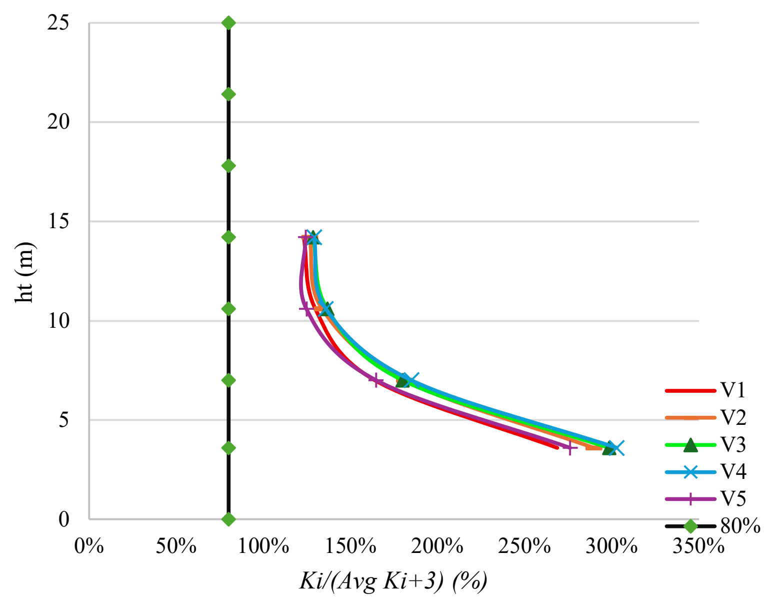

Figure 5.59 Stiffness Ratio of Three Story Above in Strong Axis Comparison

Based on Figure 5.59 and Figure 5.61 the secondary beam configuration that have most influence is model variations 3. The structure model variation 3 is the structure model with the furthest value from the standard code, 70%. However, different from the result of story stiffness ratio in weak axis. The stiffness ratio graph is denser. Thus, the secondary beam has less influence on inter story stiffness ratio in strong axis than inter story stiffness ratio in weak axis.

5.12 Accidental Torsion

Determination of accidental torsion is obtained from structure displacement at two edge point of the structure transverse to an axis. The seismic design uses equivalent lateral force analysis. Input ELF to SAP2000 can be seen in Figure 5.60

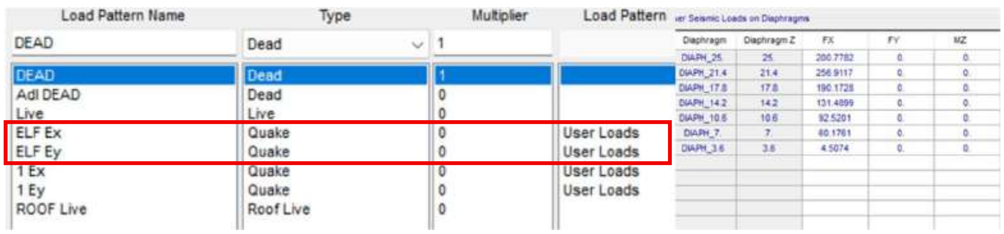

Figure 5.60 ELF Input in SAP2000

5.12.1 Accidental torsion result

The point chosen in both edge that have column, the implementation in software can be seen in Figure 5.61 and Figure 5.62.

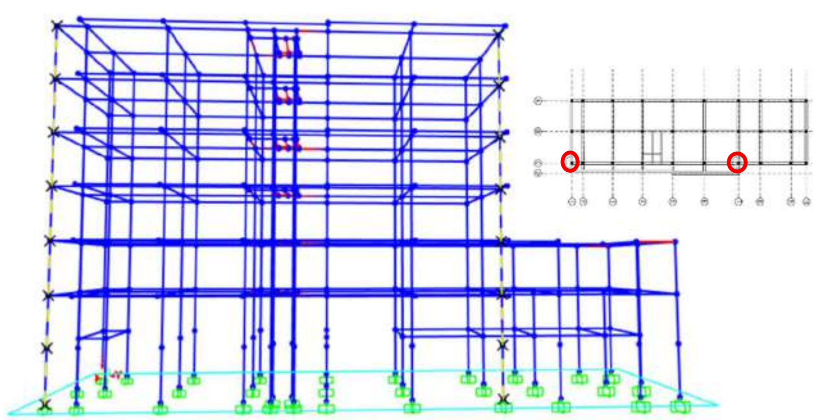

Figure 5.61 Edges Point Selection for Seismic Force in Weak axis

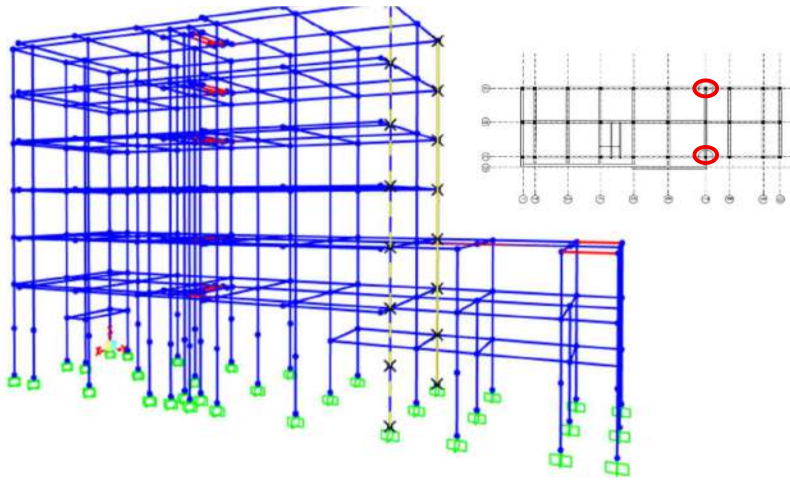

Figure 5.62 Edges Point Selection for Seismic Force in Strong axis

Structure joint displacement is obtained from the structure analysis. Example of calculation of 7th story's accidental torsion with the force in weak axis is as follows:

$\delta_{7south} = 0.047$ m

$\delta_{7north} = 0.049$ m

$\delta_{6south} = 0.044$ m

$\delta_{6Nor} = 0.044$ m

$\Delta_{7sout} = \delta_{7sout} - \delta_{6sout}$

$\Delta_{7sout} = 0.047 - 0.044$

$\Delta_{7south} = 0.003$ m

$\Delta_{7nort} = \delta_{7north} - \delta_{6nort}$

$\Delta_{7nort} = 0.049 - 0.044$

$\Delta_{7nort} = 0.005$ m

$\Delta_{7\max} = 0.005$ m

$\Delta_{7\text{average}} = \frac{\Delta_{7sout} - \Delta_{7north}}{2}$

$$\Delta_{7\text{average}} = \frac{0.003+0.005}{2}$$

$$\Delta_{7average} = 0.004 \text{ m}$$

After the drifts is obtained, the torsional irregularity ratio can calculate in accordance with equation (3.17).

$$TIR = \frac{0.005}{0.004}$$

$$TIR = 1.163$$

TIR of 7th floor is below than 1.2, the torsional irregularity does not occur. The other torsional irregularity in each story and other model is presented in Table 5.31 to Table 5.40.

Table 5.31 Torsional Irregularity in Weak Axis of Model 1

Story	δ East (m)	δ West (m)	Δ North (m)	Δ South (m)	Δ AVG (m)	ΔMax (m)	ΔMax/ΔAVG	Check
25	0.049	0.047	0.005	0.003	0.004	0.005	1.163	Regular
21.4	0.045	0.044	0.007	0.005	0.006	0.007	1.185	Regular
17.8	0.037	0.039	0.010	0.007	0.008	0.010	1.165	Regular
14.2	0.028	0.032	0.010	0.009	0.010	0.010	1.079	Regular
10.6	0.017	0.023	0.008	0.010	0.009	0.010	1.094	Regular
7	0.009	0.013	0.006	0.009	0.007	0.009	1.194	Regular
3.6	0.003	0.004	0.003	0.004	0.004	0.004	1.188	Regular

Table 5.32 Torsional Irregularity in Strong Axis of Model 1

Story	δ East (m)	δ West (m)	Δ North (m)	Δ South (m)	Δ AVG (m)	ΔMax (m)	ΔMax/ΔAVG	Check
25	0.027	0.029	0.002	0.003	0.002	0.003	1.044	Regular
21.4	0.024	0.026	0.004	0.004	0.004	0.004	1.045	Regular
17.8	0.021	0.022	0.005	0.005	0.005	0.005	1.043	Regular
14.2	0.016	0.017	0.005	0.006	0.006	0.006	1.041	Regular
10.6	0.010	0.011	0.005	0.005	0.005	0.005	1.031	Regular
7	0.005	0.005	0.003	0.004	0.004	0.004	1.034	Regular
3.6	0.002	0.002	0.002	0.002	0.002	0.002	1.015	Regular

Table 5.33 Torsional Irregularity in Weak Axis of Model 2

Story	δ East (m)	δ West (m)	Δ North (m)	Δ South (m)	Δ AVG (m)	ΔMax (m)	ΔMax/ΔAVG	Check
25	0.057	0.055	0.006	0.004	0.005	0.006	1.174	Regular
21.4	0.051	0.051	0.009	0.006	0.008	0.009	1.182	Regular
17.8	0.042	0.044	0.012	0.009	0.010	0.012	1.156	Regular
14.2	0.030	0.036	0.012	0.011	0.011	0.012	1.063	Regular
10.6	0.018	0.025	0.009	0.011	0.010	0.011	1.116	Regular
7	0.009	0.014	0.006	0.009	0.008	0.009	1.216	1a Irregular
3.6	0.003	0.005	0.003	0.005	0.004	0.005	1.220	1a Irregular

Table 5.34 Torsional Irregularity in Strong Axis of Model 2

Story	δ East (m)	δ West (m)	Δ North (m)	Δ South (m)	Δ AVG (m)	ΔMax (m)	ΔMax/ΔAVG	Check
25	0.029	0.033	0.002	0.003	0.003	0.003	1.081	Regular
21.4	0.027	0.030	0.004	0.005	0.004	0.005	1.080	Regular
17.8	0.023	0.026	0.005	0.006	0.006	0.006	1.078	Regular
14.2	0.017	0.019	0.006	0.007	0.006	0.007	1.074	Regular
10.6	0.012	0.013	0.006	0.006	0.006	0.006	1.055	Regular
7	0.006	0.006	0.004	0.004	0.004	0.004	1.063	Regular
3.6	0.002	0.002	0.002	0.002	0.002	0.002	1.005	Regular

Table 5.35 Torsional Irregularity in Weak Axis of Model 3

Story	δ East (m)	δ West (m)	Δ North (m)	Δ South (m)	Δ AVG (m)	ΔMax (m)	ΔMax/ΔAVG	Check
25	0.053	0.054	0.005	0.004	0.005	0.005	1.135	Regular
21.4	0.047	0.050	0.008	0.006	0.007	0.008	1.143	Regular
17.8	0.039	0.044	0.011	0.008	0.010	0.011	1.118	Regular
14.2	0.028	0.036	0.011	0.010	0.011	0.011	1.030	Regular
10.6	0.017	0.025	0.009	0.011	0.010	0.011	1.137	Regular
7	0.009	0.014	0.006	0.009	0.008	0.009	1.228	1a Irregular
3.6	0.003	0.005	0.003	0.005	0.004	0.005	1.227	1a Irregular

Table 5.36 Torsional Irregularity in Strong Axis of Model 3

Story	δ East (m)	δ West (m)	Δ North (m)	Δ South (m)	Δ AVG (m)	ΔMax (m)	ΔMax/ΔAVG	Check
25	0.035	0.031	0.003	0.003	0.003	0.003	1.061	Regular
21.4	0.032	0.029	0.005	0.004	0.005	0.005	1.060	Regular
17.8	0.027	0.024	0.006	0.006	0.006	0.006	1.059	Regular
14.2	0.020	0.019	0.007	0.006	0.007	0.007	1.056	Regular
10.6	0.013	0.012	0.006	0.006	0.006	0.006	1.043	Regular
7	0.007	0.006	0.004	0.004	0.004	0.004	1.053	Regular
3.6	0.002	0.002	0.002	0.002	0.002	0.002	1.013	Regular

Table 5.37 Torsional Irregularity in Weak Axis of Model 4

Story	δ East (m)	δ West (m)	Δ North (m)	Δ South (m)	Δ AVG (m)	ΔMax (m)	ΔMax/ΔAVG	Check
25	0.057	0.059	0.006	0.005	0.006	0.006	1.144	Regular
21.4	0.050	0.054	0.009	0.007	0.008	0.009	1.141	Regular
17.8	0.041	0.047	0.012	0.009	0.011	0.012	1.111	Regular
14.2	0.029	0.038	0.012	0.011	0.012	0.012	1.018	Regular
10.6	0.018	0.027	0.009	0.012	0.010	0.012	1.153	Regular
7	0.009	0.014	0.006	0.010	0.008	0.010	1.247	1a Irregular
3.6	0.003	0.005	0.003	0.005	0.004	0.005	1.252	1a Irregular

Table 5.38 Torsional Irregularity in Strong Axis of Model 4

Story	δ East (m)	δ West (m)	Δ North (m)	Δ South (m)	Δ AVG (m)	ΔMax (m)	ΔMax/ΔAVG	Check
25	0.035	0.030	0.003	0.003	0.003	0.003	1.105	Regular
21.4	0.032	0.027	0.005	0.004	0.005	0.005	1.102	Regular
17.8	0.027	0.023	0.007	0.005	0.006	0.007	1.099	Regular
14.2	0.020	0.018	0.007	0.006	0.007	0.007	1.093	Regular
10.6	0.013	0.012	0.006	0.006	0.006	0.006	1.068	Regular
7	0.007	0.006	0.004	0.004	0.004	0.004	1.075	Regular
3.6	0.002	0.002	0.002	0.002	0.002	0.002	1.009	Regular

Table 5.39 Torsional Irregularity in Weak Axis of Model 5

Story	δ East (m)	δ West (m)	Δ North (m)	Δ South (m)	Δ AVG (m)	ΔMax (m)	ΔMax/ΔAVG	Check
25	0.049	0.047	0.004	0.003	0.004	0.004	1.170	Regular
21.4	0.045	0.044	0.007	0.005	0.006	0.007	1.190	Regular
17.8	0.037	0.039	0.010	0.007	0.008	0.010	1.170	Regular
14.2	0.028	0.032	0.010	0.009	0.009	0.010	1.085	Regular
10.6	0.017	0.023	0.008	0.010	0.009	0.010	1.086	Regular
7	0.009	0.013	0.006	0.009	0.007	0.009	1.189	Regular
3.6	0.003	0.004	0.003	0.004	0.004	0.004	1.181	Regular

Table 5.40 Torsional Irregularity in Strong Axis of Model 5

Story	δ East (m)	δ West (m)	Δ North (m)	Δ South (m)	Δ AVG (m)	ΔMax (m)	ΔMax/ΔAVG	Check
25	0.032	0.029	0.003	0.002	0.002	0.003	1.063	Regular
21.4	0.030	0.027	0.004	0.004	0.004	0.004	1.061	Regular
17.8	0.026	0.023	0.006	0.005	0.005	0.006	1.058	Regular
14.2	0.020	0.018	0.007	0.006	0.006	0.007	1.055	Regular
10.6	0.013	0.012	0.006	0.006	0.006	0.006	1.042	Regular
7	0.007	0.007	0.005	0.004	0.004	0.005	1.052	Regular
3.6	0.002	0.002	0.002	0.002	0.002	0.002	1.021	Regular

5.12.2 Accidental torsion discussion

Torsional irregularity analysis indicates the accidental torsion of the structure. Value that does not exceed the standard, 1.2 (BSN, 2019a). The comparison result of torsional irregularity analysis can be seen in Figure 5.63 and Figure 5.64.

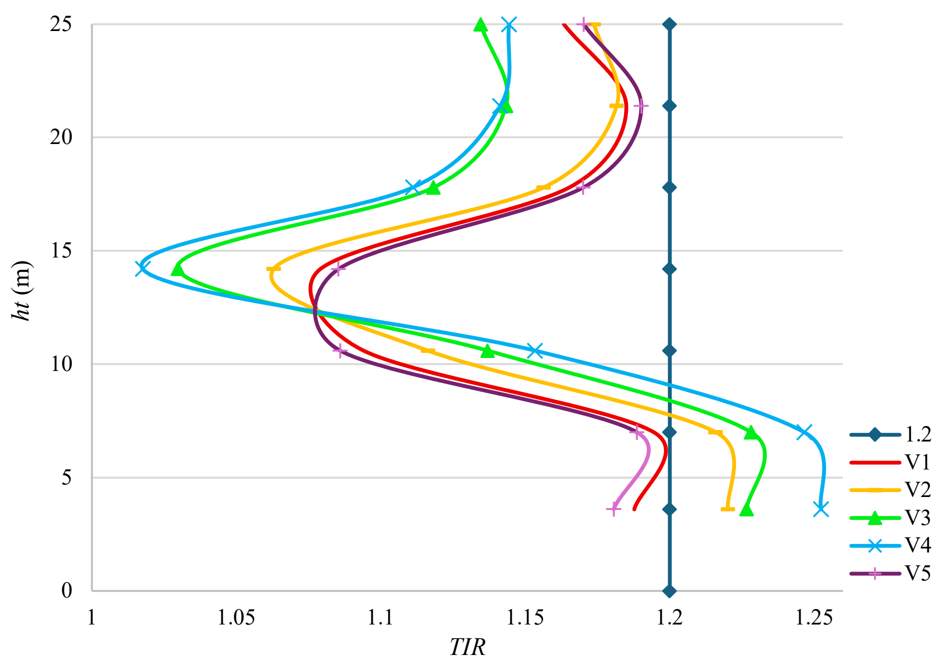

Figure 5.63 Accidental Torsion in Weak Axis

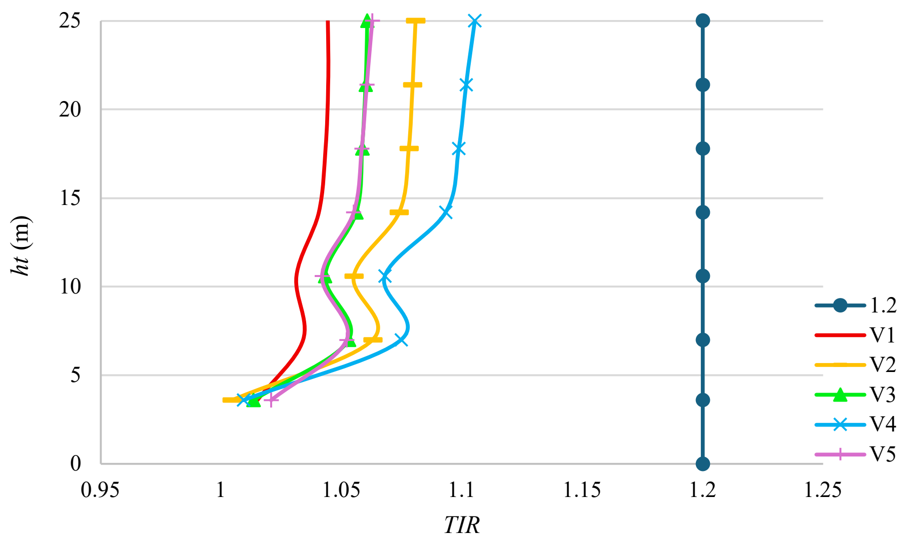

Figure 5.64 Accidental Torsion in Strong Axis

Based on Figure 5.63 the secondary beam configuration that have most influence is model variations 5 because structure model variation 5 is the structure model that does not exceed 1.2 in torsion irregularity in weak axis. Torsion irregularity ratio can predict the absolute collapse capacity (ACMR), when the torsion irregularity ratio is exceeded 1.2 the absolute collapse capacity (ACMR) will drastically decrease (Jared, 2011). The model variation with secondary beam configuration 5 will have highest absolute collapse capacity among of the rest model variations.

Based on Figure 5.64 the secondary beam configuration that have most influence is model variations 3 and 5. The structure model variation 3 and 5 is the structure model with the furthest value from the upper boundary, 1.2. However, different from the result of torsion irregularity in weak axis. Each model variations do not limit the upper boundary, 1.2. Thus, the secondary beam has less influence on torsion irregularity analysis in strong axis than torsion irregularity analysis in weak axis.

5.13 Irregularity Assessment

5.13.1 Vertical irregularities Type 1

Based on the section 5.11 there are no inter-story stiffness ratio lower than 70% or ratio of 80% average three-story stiffness above. Furthermore, there are no type 1a and 1b vertical irregularity.

5.13.2 Vertical irregularities Type 2

Analysis of mass irregularity is calculated based on manual calculations of each model variation. The inter story above and below ratio should not exceed 150% (BSN, 2019a), analysis result of mass irregularity can be seen in Table 5.41.

Table 5.41 Mass Irregularity Analysis

Story level	V1		V2		V3		V4		V5	
	W_i/W_{i+1}	W_i/W_{i-1}	W_i/W_{i+1}	W_i/W_{i-1}	W_i/W_{i+1}	W_i/W_{i-1}	W_i/W_{i+1}	W_i/W_{i-1}	W_i/W_{i+1}	W_i/W_{i-1}
7		61%		60%		60%		60%		63%
6	165%	100%	167%	100%	166%	100%	167%	100%	159%	100%
5	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
4	100%	88%	100%	85%	100%	85%	100%	85%	100%	86%
3	114%	79%	118%	81%	118%	81%	118%	81%	116%	82%
2	127%	448%	124%	450%	124%	441%	124%	455%	122%	457%
1.5	22%	137%	22%	134%	23%	138%	22%	133%	22%	140%
1	73%		74%		73%		75%		71%	

Based on Table 5.41 the highlighted value is the inter story mass ratio that exceed 150%. The influence of secondary beam applications is caused by structural gravitational forces, i.e. structural dead load and additional dead load.

5.13.3 Vertical irregularities Type 3

Analysis of vertical geometry irregularity is measured based on structure shape profile. The side view of structure can be seen in Figure 5.2 while the d_i is distance between columns edge. Inter story ratio of seismic force resisting system horizontal dimension should not exceed 130% (BSN, 2019a), analysis result of vertical geometry irregularity can be seen in Table 5.42.

Table 5.42 Vertical Geometry Irregularity Analysis

Story	Strong Axis	Weak Axis
	d_i/d_{i+1}	d_i/d_{i+1}
7		100%
6	100%	100%
5	100%	100%
4	100%	100%
3	100%	100%
2	151%	100%
1.5	100%	

Based on Table 5.42 the highlighted value is the inter story ratio of seismic force resisting system horizontal dimension that exceed 130%. The secondary beam has no influence in vertical geometry irregularity analysis.

5.13.4 Vertical irregularities Type 4

Structure plane discontinuity irregularities in lateral force-bearing vertical elements does not occur in the structure plane. The secondary beam has no influence in vertical irregularity type 4 analysis.

5.13.5 Vertical irregularities Type 5

Analysis of soft story irregularity due to discontinuity at story lateral strength can be calculated with inter story cumulative vertical distribution factor ratio. The vertical distribution factor (C_{vx}) obtained from section 5.9. Analysis result of soft story lateral strength can be seen in Table 5.43.

Table 5.43 Soft Story Lateral Strength Irregularity Analysis

h_t (m)	V_{xi}/V_{xi+1}				
	V1	**V2**	**V3**	**V4**	**V5**
25					
21.4	228%	228%	228%	228%	223%
17.8	142%	141%	141%	141%	141%
14.2	120%	120%	120%	120%	120%
10.6	112%	112%	112%	112%	112%
7	107%	106%	107%	106%	107%
3.6	100%	100%	100%	100%	100%
0	0%	0%	0%	0%	0%

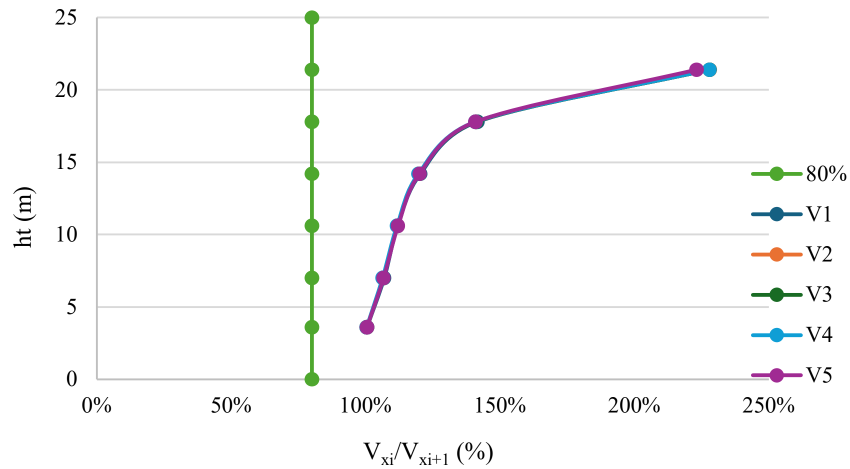

Figure 5.65 Soft Story Lateral Strength Irregularity Comparison

Based on the Figure 5.65 each model variations story lateral strength is not lower than 80% (BSN, 2019a). The story lateral strength graph is very dense. Hence it implies that the influence of secondary beam is subtle.

5.13.6 Horizontal irregularities Type 1

Based on the section 5.12 there are torsional irregularity ratio more than 1.2 in model variation 2, 3, and 4. Consequently, the structure that exceed torsional irregularity ratio limit need to be amplified for the structure eccentricity. However, since the research focus on the secondary beam influences only, the amplified factor for torsion is neglect. As a result, model variation 2, 3, and 4 have horizontal irregularity type 1a.

5.13.7 Horizontal irregularities Type 2

The building plan has symmetric shape, building plan does not have interior corner. As a result, the structure does not have horizontal irregularity type 2.

5.13.8 Horizontal irregularities Type 3

Building plan diaphragm area is 455 m^2. While the void of building is consisting of lift and emergency stair with area of 13.5 m^2. The ratio of overall diaphragm area to void area is 3%. As a result, the building does not have horizontal irregularity type 3.

5.13.9 Horizontal irregularities Type 4

Building plan configuration has symmetrically vertical column. As a result, the building does not have horizontal irregularity type 4.

5.13.10 Horizontal irregularities Type 5

Building plan configuration is symmetric and parallel frame. As a result, the building does not have horizontal irregularity type 5.

5.13.11 Irregularities assessment result

The irregularities assessment of every model variation is recapitulated in Table 5.44 for horizontal irregularity analysis and Table 5.45 for vertical irregularity analysis.

Table 5.44 Horizontal Irregularity Analysis Recapitulation

Type	V1	V2	V3	V4	V5
1	Regular	1a	1a	1a	Regular
2	Regular	Regular	Regular	Regular	Regular
3	Regular	Regular	Regular	Regular	Regular
4	Regular	Regular	Regular	Regular	Regular
5	Regular	Regular	Regular	Regular	Regular

Table 5.45 Vertical Irregularity Analysis Recapitulation

Type	V1	V2	V3	V4	V5
1	Regular	Regular	Regular	Regular	Regular
2	Irregular	Irregular	Irregular	Irregular	Irregular
3	Irregular	Irregular	Irregular	Irregular	Irregular
4	Regular	Regular	Regular	Regular	Regular
5	Regular	Regular	Regular	Regular	Regular

Based on the Table 5.44 and Table 5.45 the structure should be analysed using Modal Response Spectrum Analysis in accordance with Table 3.8 Allowed Analysis Procedure.

5.14 Story drift

Story drift determination should use inter-story drift from response spectrum analysis procedure as the consequences of irregularities analysis in sections 5.13. The drift is taken from the vertical projection point in structures centre of mass.

5.14.1 Story drift result

Determination of centre mass in structural analysis program should define new special joint, the special joint in structural analysis program can be seen in Figure 5.66.

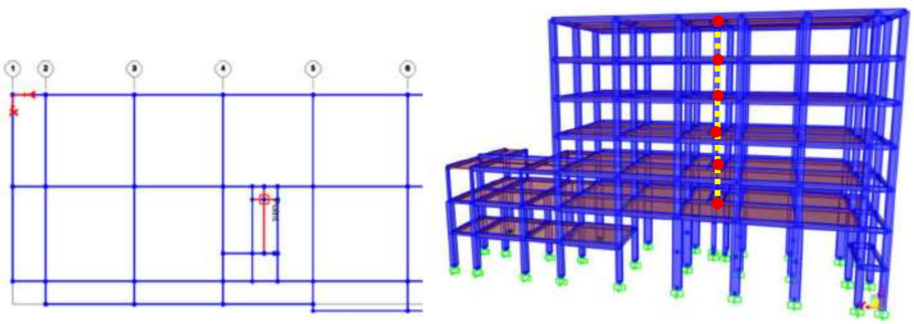

Figure 5.66 Special Joint at Centre Mass

Allowable story drift for special moment resisting reinforced concrete frame with risk category II is $0.020h_t$ in accordance with (BSN, 2019a). The calculation example of 7th story drift in weak axis is as follows:

$$\delta_{xe\,7} = 0.033 \text{ m}$$

$$\delta_{xe\,6} = 0.031 \text{ m}$$

$$\delta_{x\,7} = \frac{5.5 \times 0.033}{1} \times 1$$

$$\delta_{x\,7} = 0.183 \text{ m}$$

$$\delta_{x\,6} = \frac{5.5 \times 0.031}{1} \times 1 \text{ m}$$

$$\delta_{x\,6} = 0.170 \text{ m}$$

$$\Delta_7 = \delta_7 - \delta_6$$

$$\Delta_7 = 0.183 - 0.170 \text{ m}$$

$$\Delta_7 = 0.013 \text{ m}$$

$$\frac{\Delta_{\text{all}}}{\rho} = \frac{0.020 \times 3.6}{1}$$

$$\frac{\Delta_{\text{all}}}{\rho} = 0.072 \text{ m}$$

Story drift, Δ of 7th floor is below than Δ_{all}/ρ, 0.072 m. Thus, the story drift is allowable to occur. The other story drift in each story and other model is presented in Table 5.46 and Table 5.55.

Table 5.46 Story Drift in Weak Axis of Model 1

Story height	height	δ_{xe}	δ_X	Δ	Δ_{allow}	$\frac{\Delta allow}{\rho}$	Status
m	m	m	m	m	m	m	
25	3.6	0.033	0.183	0.013	0.072	0.072	OK
21.4	3.6	0.031	0.170	0.021	0.072	0.072	OK
17.8	3.6	0.027	0.148	0.029	0.072	0.072	OK
14.2	3.6	0.022	0.120	0.035	0.072	0.072	OK
10.6	3.6	0.015	0.085	0.037	0.072	0.072	OK
7	3.4	0.009	0.048	0.048	0.068	0.068	OK

Table 5.47 Story Drift in Strong Axis of Model 1

Story height	height	δ_{xe}	δ_X	Δ	Δ_{allow}	$\frac{\Delta allow}{\rho}$	Status
m	m	m	m	m	m	m	
25.0	3.6	0.028	0.155	0.011	0.072	0.072	OK
21.4	3.6	0.026	0.144	0.019	0.072	0.072	OK
17.8	3.6	0.023	0.125	0.026	0.072	0.072	OK
14.2	3.6	0.018	0.099	0.031	0.072	0.072	OK
10.6	3.6	0.012	0.068	0.031	0.072	0.072	OK
7.0	3.4	0.007	0.037	0.037	0.068	0.068	OK

Table 5.48 Story Drift in Weak Axis of Model 2

Story height	height	δ_{xe}	δ_X	Δ	Δ_{allow}	$\frac{\Delta allow}{\rho}$	Status
m	m	m	m	m	m	m	
25	3.6	0.038	0.212	0.018	0.072	0.072	OK
21.4	3.6	0.035	0.194	0.027	0.072	0.072	OK
17.8	3.6	0.030	0.167	0.035	0.072	0.072	OK
14.2	3.6	0.024	0.132	0.040	0.072	0.072	OK
10.6	3.6	0.017	0.091	0.041	0.072	0.072	OK
7	3.4	0.009	0.050	0.050	0.068	0.068	OK

Table 5.49 Story Drift in Strong Axis of Model 2

Story height	height	δ_{xe}	δ_X	Δ	Δ_{allow}	$\frac{\Delta allow}{\rho}$	Status
m	m	m	m	m	m	m	
25.0	3.6	0.029	0.162	0.013	0.072	0.072	OK
21.4	3.6	0.027	0.150	0.021	0.072	0.072	OK
17.8	3.6	0.023	0.129	0.028	0.072	0.072	OK
14.2	3.6	0.018	0.101	0.032	0.072	0.072	OK
10.6	3.6	0.012	0.068	0.032	0.072	0.072	OK
7.0	3.4	0.007	0.036	0.036	0.068	0.068	OK

Table 5.50 Story Drift in Weak Axis of Model 3

Story height	height	δ_{xe}	δ_X	Δ	Δ_{allow}	$\frac{\Delta allow}{\rho}$	Status
m	m	m	m	m	m	m	
25	3.6	0.037	0.203	0.016	0.072	0.072	OK
21.4	3.6	0.034	0.187	0.025	0.072	0.072	OK
17.8	3.6	0.029	0.162	0.033	0.072	0.072	OK
14.2	3.6	0.023	0.129	0.039	0.072	0.072	OK
10.6	3.6	0.016	0.090	0.040	0.072	0.072	OK
7	3.4	0.009	0.050	0.050	0.068	0.068	OK

Table 5.51 Story Drift in Strong Axis of Model 3

Story height	height	δ_{xe}	δ_X	Δ	Δ_{allow}	$\frac{\Delta allow}{\rho}$	Status
m	m	m	m	m	m	m	
25.0	3.6	0.030	0.166	0.013	0.072	0.072	OK
21.4	3.6	0.028	0.152	0.021	0.072	0.072	OK
17.8	3.6	0.024	0.131	0.029	0.072	0.072	OK
14.2	3.6	0.019	0.102	0.033	0.072	0.072	OK
10.6	3.6	0.013	0.069	0.032	0.072	0.072	OK
7.0	3.4	0.007	0.036	0.036	0.068	0.068	OK

Table 5.52 Story Drift in Weak Axis of Model 4

Story height	height	δ_{xe}	δ_X	Δ	Δ_{allow}	$\frac{\Delta allow}{\rho}$	Status
m	m	m	m	m	m	m	
25	3.6	0.040	0.219	0.019	0.072	0.072	OK
21.4	3.6	0.036	0.200	0.028	0.072	0.072	OK
17.8	3.6	0.031	0.172	0.037	0.072	0.072	OK
14.2	3.6	0.025	0.136	0.042	0.072	0.072	OK
10.6	3.6	0.017	0.094	0.043	0.072	0.072	OK
7	3.4	0.009	0.051	0.051	0.068	0.068	OK

Table 5.53 Story Drift in Strong Axis of Model 4

Story height	height	δ_{xe}	δ_X	Δ	Δ_{allow}	$\frac{\Delta allow}{\rho}$	Status
0	m	m	m	m	m	m	
25.0	3.6	0.030	0.167	0.014	0.072	0.072	OK
21.4	3.6	0.028	0.153	0.022	0.072	0.072	OK
17.8	3.6	0.024	0.132	0.029	0.072	0.072	OK
14.2	3.6	0.019	0.102	0.034	0.072	0.072	OK
10.6	3.6	0.013	0.069	0.033	0.072	0.072	OK
7.0	3.4	0.007	0.036	0.036	0.068	0.068	OK

Table 5.54 Story Drift in Weak Axis of Model 5

Story height	height	δ_{xe}	δ_X	Δ	Δ_{allow}	$\frac{\Delta allow}{\rho}$	Status
m	m	m	m	m	m	m	
25	3.6	0.034	0.186	0.013	0.072	0.072	OK
21.4	3.6	0.031	0.173	0.021	0.072	0.072	OK
17.8	3.6	0.028	0.151	0.029	0.072	0.072	OK
14.2	3.6	0.022	0.122	0.035	0.072	0.072	OK
10.6	3.6	0.016	0.087	0.038	0.072	0.072	OK
7	3.4	0.009	0.050	0.050	0.068	0.068	OK

Table 5.55 Story Drift in Strong Axis of Model 5

Story height	height	δ_{xe}	δ_X	Δ	Δ_{allow}	$\frac{\Delta allow}{\rho}$	Status
m	m	m	m	m	m	m	
25.0	3.6	0.028	0.156	0.011	0.072	0.072	OK
21.4	3.6	0.026	0.145	0.019	0.072	0.072	OK
17.8	3.6	0.023	0.126	0.026	0.072	0.072	OK
14.2	3.6	0.018	0.100	0.031	0.072	0.072	OK
10.6	3.6	0.013	0.069	0.032	0.072	0.072	OK
7.0	3.4	0.007	0.038	0.038	0.068	0.068	OK

5.14.2 Story drift discussion

Story drift analysis result of each model variation is compared in Figure 5.67 and Figure 5.68. Value that does not exceed the standard, $0.020h_t$ (BSN, 2019a), indicates the structure drift is safe and allowable to occurs.

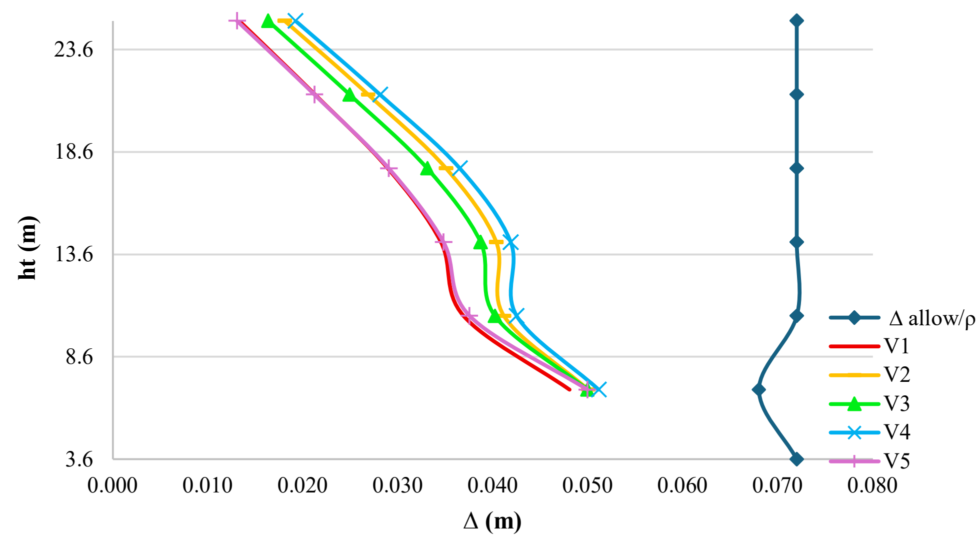

Figure 5.67 Story Drift in Weak Axis Comparison

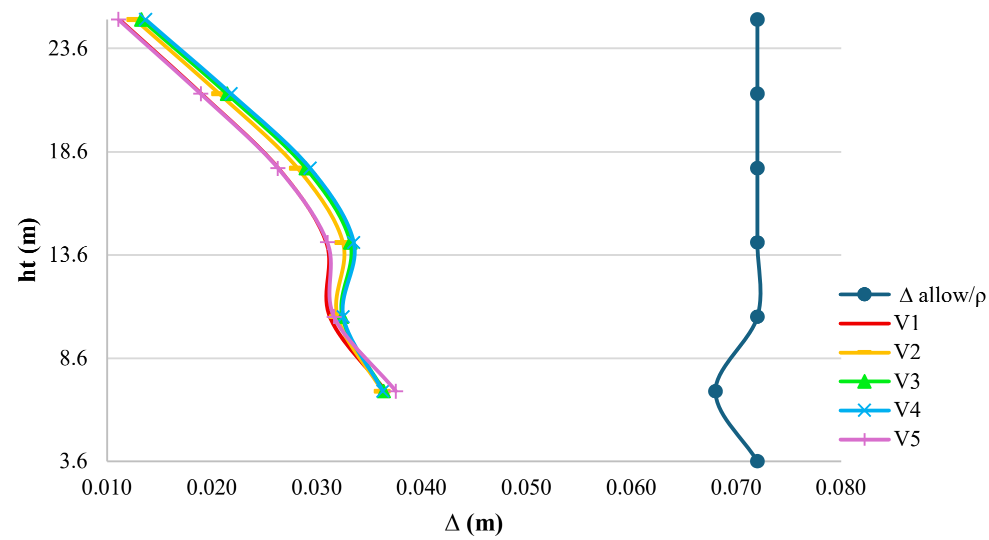

Figure 5.68 Story Drift in Strong Axis Comparison

Based on Figure 5.67 the secondary beam configuration that have most influence is model variations 5 because story drift value in weak axis is the furthest to the regulation (BSN, 2019a).

Research proposed by (Naem, 2000) stated that the lateral stiffness increases, the load of the building will increase, and the stability problem chances can be reduced. Based on the relation of previous section 5.9 and story drift analysis result is in accordance with that research statement. As a result, the structure with secondary beam configuration 5 with most base shear and lateral force has least story drift. Thus, the secondary beam configuration 5 have influence to reduce story drift and reduce stability problem.

Furthermore, the comparison result of story drift analysis in strong axis can be seen in Figure 5.68. Different from the result of story drift in weak axis the secondary beam has less influence on story drift in strong axis.

5.15 P-delta Effect

P-delta effect is the relation of vertical story section design and story drift defined in section 5.14 over story section seismic shear force and story height moment. Value of the story vertical design load is obtained from software analysis program section cut analysis with service load applied. While, story seismic shear force is obtained from structural analysis program section cut analysis with seismic design force, i.e. response spectrum analysis procedure.

5.15.1 P-delta effect analysis result

The calculation of 7th story's P-delta effect in weak axis is in accordance with equation (3.19) to (3.20) as follows.

$$\theta_7 = \frac{P_{x7}\Delta_7 I_e}{V_{x7}h_7 C_d}$$

$$\theta_7 = \frac{3059.251\times0.013\times1}{159.319\times3.6\times5.5}$$

$$\theta_7 = 0.013$$

$$\theta_{7ma} = \frac{0.5}{1\times5.5}$$

$$\theta_{7max} = 0.091$$

Stability coefficient, θ of 7th floor when seismic design in weak axis applied is below than maximum stability coefficient, $\theta_{\max}$. Thus, the P-delta effect is no need to consider. The other P-delta effect analysis in each story and other model is presented in the Table 5.56 to Table 5.65.

Table 5.56 P-delta Effect in Weak Axis of Model 1

Height	Story	P_x	V_x	Δ	θ	θ_{max}	Status
m		kN	kN	m			
3.6	7	3059.251	159.319	0.013	1.291%	9.091%	Ok
3.6	6	7747.033	336.738	0.021	2.473%	9.091%	Ok
3.6	5	12434.815	449.218	0.029	4.048%	9.091%	Ok
3.6	4	17122.597	530.083	0.035	5.633%	9.091%	Ok
3.6	3	22660.030	620.321	0.037	6.812%	9.091%	Ok
3.4	2	29190.078	719.828	0.048	10.421%	9.091%	Not ok
3.6	1.5	31621.977	741.014	0.000	0.000%	9.091%	Ok

Table 5.57 P-delta Effect in Strong Axis of Model 1

Height	Story	P_x	V_x	Δ	θ	θ_{max}	Status
m		kN	kN	m			
3.6	7	3059.251	200.818	0.011	0.859%	9.091%	Ok
3.6	6	7747.033	436.983	0.019	1.700%	9.091%	Ok
3.6	5	12434.815	606.345	0.026	2.726%	9.091%	Ok
3.6	4	17122.597	733.128	0.031	3.645%	9.091%	Ok
3.6	3	22660.030	857.337	0.031	4.169%	9.091%	Ok
3.4	2	29190.078	966.168	0.037	5.903%	9.091%	Ok
3.6	1.5	31621.977	988.348	0.000	0.000%	9.091%	Ok

Table 5.58 P-delta Effect in Weak Axis of Model 2

Height	Story	P_x	V_x	Δ	θ	θ_{max}	Status
m		kN	kN	m			
3.6	7	2982.171	160.175	0.018	1.701%	9.091%	Ok
3.6	6	7592.872	325.531	0.027	3.165%	9.091%	Ok
3.6	5	12203.574	420.041	0.035	5.146%	9.091%	Ok
3.6	4	16814.276	487.683	0.040	7.027%	9.091%	Ok
3.6	3	22274.629	575.677	0.041	8.047%	9.091%	Ok
3.4	2	28727.596	678.587	0.050	11.328%	9.091%	Not ok
3.6	1.5	31147.503	700.574	0.000	0.000%	9.091%	Ok

Table 5.59 P-delta Effect in Strong Axis of Model 2

Height	Story	P_x	V_x	Δ	θ	θ_{max}	Status
m		kN	kN	m			
3.6	7	2982.171	194.598	0.013	0.980%	9.091%	Ok
3.6	6	7592.872	417.959	0.021	1.899%	9.091%	Ok
3.6	5	12203.574	573.177	0.028	3.027%	9.091%	Ok
3.6	4	16814.276	688.240	0.032	4.008%	9.091%	Ok
3.6	3	22274.629	804.503	0.032	4.468%	9.091%	Ok
3.4	2	28727.596	909.872	0.036	6.118%	9.091%	Ok
3.6	1.5	31147.503	931.664	0.000	0.000%	9.091%	Ok

Table 5.60 P-delta Effect in Weak Axis of Model 3

Height	Story	P_x	V_x	Δ	θ	θ_{max}	Status
m		kN	kN	m			
3.6	7	3009.752	159.848	0.016	1.555%	9.091%	Ok
3.6	6	7648.035	328.979	0.025	2.927%	9.091%	Ok
3.6	5	12286.318	428.894	0.033	4.788%	9.091%	Ok
3.6	4	16924.601	500.050	0.039	6.614%	9.091%	Ok
3.6	3	22412.535	588.083	0.040	7.741%	9.091%	Ok
3.4	2	28893.084	689.771	0.050	11.190%	9.091%	Not ok
3.6	1.5	31318.893	711.450	0.000	0.000%	9.091%	Ok

Table 5.61 P-delta Effect in Strong Axis of Model 3

Height	Story	P_x	V_x	Δ	θ	θ_{max}	Status
		kN	kN	m			
3.6	7	3009.752	196.160	0.013	1.028%	9.091%	Ok
3.6	6	7648.035	417.148	0.021	1.984%	9.091%	Ok
3.6	5	12286.318	569.017	0.029	3.156%	9.091%	Ok
3.6	4	16924.601	681.412	0.033	4.164%	9.091%	Ok
3.6	3	22412.535	796.218	0.032	4.609%	9.091%	Ok
3.4	2	28893.084	901.373	0.036	6.233%	9.091%	Ok
3.6	1.5	31318.893	923.154	0.000	0.000%	9.091%	Ok

Table 5.62 P-delta Effect in Weak Axis of Model 4

Height	Story	P_x	V_x	Δ	θ	θ_{max}	Status
m		kN	kN	m			
3.6	7	2972.457	160.709	0.019	1.797%	9.091%	Ok
3.6	6	7575.014	323.937	0.028	3.324%	9.091%	Ok
3.6	5	12177.570	415.498	0.037	5.404%	9.091%	Ok
3.6	4	16780.127	481.250	0.042	7.374%	9.091%	Ok
3.6	3	22232.334	569.399	0.043	8.384%	9.091%	Ok
3.4	2	28677.157	673.345	0.051	11.648%	9.091%	Not ok
3.6	1.5	31093.379	695.483	0.000	0.000%	9.091%	Ok

Table 5.63 P-delta Effect in Strong Axis of Model 4

Height	Story	P_x	V_x	Δ	θ	θ_{max}	Status
		kN	kN	m			
3.6	7	2972.457	193.143	0.014	1.061%	9.091%	Ok
3.6	6	7575.014	411.086	0.022	2.032%	9.091%	Ok
3.6	5	12177.570	560.032	0.029	3.223%	9.091%	Ok
3.6	4	16780.127	669.547	0.034	4.242%	9.091%	Ok
3.6	3	22232.334	782.441	0.033	4.665%	9.091%	Ok
3.4	2	28677.157	887.125	0.036	6.270%	9.091%	Ok
3.6	1.5	31093.379	908.487	0.000	0.000%	9.091%	Ok

Table 5.64 P-delta Effect in Weak Axis of Model 5

Height	Story	P_x	V_x	Δ	θ	θ_{max}	Status
m		kN	kN	m			
3.6	7	3332.185	174.556	0.013	1.261%	9.091%	Ok
3.6	6	8292.902	364.047	0.021	2.441%	9.091%	Ok
3.6	5	13253.618	486.894	0.029	3.992%	9.091%	Ok
3.6	4	18214.335	576.254	0.035	5.555%	9.091%	Ok
3.6	3	24024.702	672.990	0.038	6.770%	9.091%	Ok
3.4	2	30827.684	776.516	0.050	10.603%	9.091%	Not ok
3.6	1.5	33291.705	797.478	0.000	0.000%	9.091%	Ok

Table 5.65 P-delta Effect in Strong Axis of Model 5

Height	Story	P_x	V_x	Δ	θ	θ_{max}	Status
m		kN	kN	m			
3.6	7	3332.185	216.864	0.011	0.858%	9.091%	Ok
3.6	6	8292.902	463.382	0.019	1.711%	9.091%	Ok
3.6	5	13253.618	640.750	0.026	2.745%	9.091%	Ok
3.6	4	18214.335	773.603	0.031	3.686%	9.091%	Ok
3.6	3	24024.702	903.084	0.032	4.252%	9.091%	Ok
3.4	2	30827.684	1016.443	0.038	6.086%	9.091%	Ok
3.6	1.5	33291.705	1038.354	0.000	0.000%	9.091%	Ok

5.15.2 P-delta effect discussion

P-delta effect analysis result based on Table 5.56 to Table 5.65 is compared in Figure 5.69 and Figure 5.70. The value that does not exceed the standard P-delta effect, θ_{max} (BSN, 2019a), indicates that the structure P-delta effect not effecting structure.

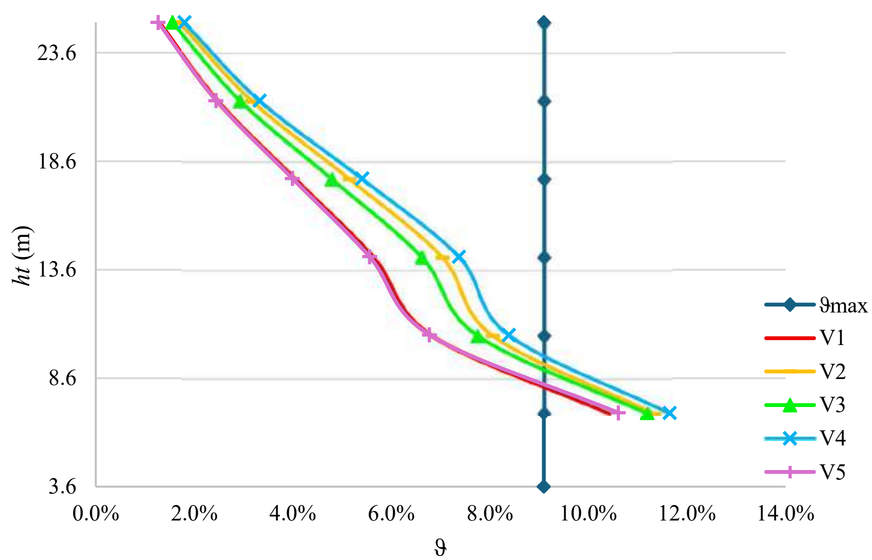

Figure 5.69 Comparison of P-delta Effect Result in Weak Axis

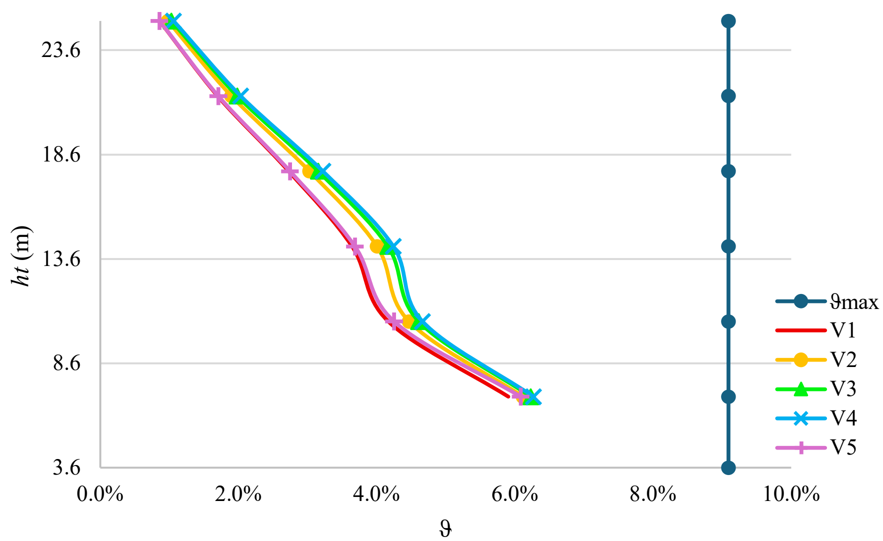

Figure 5.70 Comparison of P-delta Effect Result in Strong Axis

Based on the Figure 5.69 every model variation exceeds the upper limit. Thus, the secondary beam configuration that have most influence is model variations 5 because P-delta effect analysis in weak axis has the least far from the regulation (BSN, 2019a).

Research proposed by (Paulay, 1978) stated that the P-delta effect can increase fundamental period of a frame. Consequently, resulting large displacement will cause stiffness degradation in reinforced concrete frame. As a result, the frame exhibit strength degradation in the loading direction that could leading to incremental collapse, during subsequent ground excitations. The statement from that research is in line based on the section 5.8, the relation of fundamental period, structure stiffness and P-delta analysis result. The structure model variation 5 with least structure period has most structure stiffness and the P-delta effect is decrease among other model variation. As a result, applications of secondary beam configuration 5 could increase the structure strength.

Furthermore, the comparison result of P-delta effect analysis in strong axis can be seen in Figure 5.70. Different from the result of story drift in weak axis the secondary beam has less influence on P-delta effect in strong axis

CHAPTER VI
CONCLUSION AND RECOMMENDATION

6.1 Conclusion

Based on the comparison of each model without and with secondary beam configuration, the conclusion obtained as follows.

1. Application of secondary beams will affect the fundamental period, structure stiffness and structure deflection. The Configuration on both weak and strong axes increases structure period while the configuration on both diagonal axes decreases the structure period. The higher structure period has relation on structure stiffness reduction and lateral strength reduction and vice versa for lower structure period.
2. Application of secondary beams in buildings only influence significantly on horizontal irregularities type 1 since the other types of irregularity not affected by secondary beam application.
3. Application of secondary beams in buildings influence the lateral stiffness that related to structure story drift. The increase of lateral stiffness reduces the story drift. Thus, the secondary beam on both diagonal axes has the least story drift while the secondary beam on both weak and strong axes has most story drift.
4. Application of secondary beams in buildings can affect the stiffness of the structure and have a relationship to structure P-delta effect. The configuration on both diagonal axes has least P-delta effect because has most stiffness. While the configuration on both weak and strong axes has most P-delta effect because has least stiffness.
5. Application of secondary beams in buildings can make the structure more efficient volume even though it is not significant. The configuration on both diagonal axes has most volume while the configuration on both weak and strong axis has least volume

6. Application of secondary beam configuration in the diagonal axis is secondary beam with the most influence on structure response. secondary beam configuration in the diagonal axis increases structure stiffness and strength. However, the efficiency and implementation on the project site are unsatisfactory.
7. Application of secondary beam configuration in the weak axis is preferred if the building shape is rectangular. This is due to its efficiency and satisfactory structural response.

6.2 Recommendation

Based on the analysis and result of the final project research, the recommendation for future research is as follows.

1. Research on the same topic with larger, taller and/or non-symmetrical building plans.
2. Further research by considering reinforcement design.
3. Research that examines the collapse pattern of floor slabs without secondary beams and with the collapse pattern of floor slabs with secondary beams.

BIBLIOGRAPHY

ACI Committee 314. (2016). *314R-16 Guide to Simplified Design for Reinforced Concrete Buildings (For Buildings of Limited Size and Height, based on ACI 318-14 and ACI IPS-1, 'Essential Requirements for Reinforced Concrete Buildings')*. www.concrete.org

ASCE. (2021). *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. American Society of Civil Engineers. https://doi.org/10.1061/9780784415788

Baek, R. K. (2016). *Response of Reinforced Concrete Beams with Indirect Loading* [Thesis]. McGill University.

BSN. (2019a). *SNI 1726-2019: Tata Cara Perencanaan Ketahanan Gempa untuk Struktur Bangunan Gedung dan Nongedung*.

BSN. (2019b). *SNI 2847-2019 Persyaratan Beton Struktural untuk Bangunan Gedung*.

Budiyawan, M., & Yulandaru, P. (2006). *Pengaruh Variasi Peletakan Balok Anak Terhadap Volume Struktur Pelat dan Balok*. Universitas Islam Indonesia.

Jared. (2011). *Evaluation of the Accidental Torsion Requirement in ASCE 7 by the FEMA P695 Methodology*.

Kumar, G. A. (1990). *Response Spectrum Method in Seismic Analysis and Design of Structures*. Blackwell Scientific Publication.

Naem, F. (2000). *Chapter 07- Design for Drift and Lateral Stability*.

National Institute of Building Sciences Building Seismic Safety Council. (2012). *FEMA P-751: 2009 NEHRP Recommended Seismic Provisions: Design Examples* (T. M. Valley, Ed.; 2009th ed.).

Paulay, T. (1978). A Consideration of P-delta Effects in Ductile Reinforced Concrete Frames. *Bulletin The New Zealand National Society for Earthquake Engineering*, *11*, 151–160.

Paulay, T., & Priestley, M. J. N. (1992). *Seismic Design of Reinforced Concrete and Masonry Buildings*. A Wiley Interscience Publication.

Pawirodikromo, W. (2012). *Seismologi Teknik & Rekayasa Kegempaan*. Pustaka Pelajar.

Rakuasa, H., & Pakniany, Y. (2024). Assessing the Probability of Megathrust Earthquakes in Indonesia: A Review of Seismic Hazard Assessment and Mitigation Strategies. *Scientia Naturalis Journal of Loomingulisus Ja Innovatsioon*, *1*(3), 98–116. https://doi.org/10.70177/innovatsioon.v1i3.1275

Shuraim, B. A. (2003). Applicability of Code Design Methods to RC Slabs on Secondary Beams. Part II: Moment Variations. *King Saud Univ*, *15*(2), 199–216.

Uzodimma, O. U. (2016). Analysis and Design of a Network of Interacting Primary and Secondary Beams as Alternatives in Large Span Construction. *Ranks Michael Enterprises*.

Vijayanarayanan, A. R., Goswami, R., & Murty, R. V. C. (2017, January 9). Estimation of Storey Stiffness in Multi-storey Building. *16th World Conference on Earthquake*.

Widiyantoro, S., Gunawan, E., Muhari, A., Rawlinson, N., Mori, J., Hanifa, N. R., Susilo, S., Supendi, P., Shiddiqi, H. A., Nugraha, A. D., & Putra, H. E. (2020). Implications for Megathrust Earthquakes and Tsunamis from Seismic Gaps South of Java Indonesia. *Scientific Reports*, *10*(1), 15274. https://doi.org/10.1038/s41598-020-72142-z

Wight, J. K. (2016). *Reinforced Concrete Mechanics and Design* (H. Stark, Ed.; 7th ed.). Pearson.

Yanita, R., & Fajarwati, S. (2021). *Efektifitas Penambahan Balok Anak Terhadap Volume Material Konstruksi Pelat Lantai Beton Bertulang*. 216–221.