CHAPTER III

BASIC THEORY AND METHOD

3.1 <u>Required Strength *(U)*</u>

Based on SNI 1726:2012 combination for earthquake load that required live load, should be considered as follows:

$$E = E_h + E_v$$

Combination for earthquake load that required non-live load, should be considered as follows:

$$E = E_h - E_v$$

where

E = seismic load effect

E_h = effect of horizontal seismic forces

E_v = effect of vertical seismic forces

Horizontal seismic effect E_h determined as follows:

$$E_h = \rho Q_E$$

Where

ρ = redundancy factor

Q_E = effects of horizontal seismic forces

Horizontal seismic effect E_v determined as follows:

$$E_v = 0{,}2 S_{DS} D$$

S_{DS} = design spectral response acceleration parameter at short

D = effect of dead load

Design category seismic C according to SNI 1726:2012 section 7.5 the load effect for one direction is 100% and minimum 30% for orthogonal forces. So combination that taken in this seismic design as follows:

$$1.4\ D \tag{3-1}$$

$$1.2\ D + 1.6\ L \tag{3-2}$$

$$(1.2 + 0{,}2\ S_{DS})\ D + \rho Ex + 0.3\ \rho Ey + 1.0\ L \tag{3-3}$$

$$(1.2 + 0{,}2\ S_{DS})\ D + \rho Ex - 0.3\ \rho Ey + 1.0\ L \tag{3-4}$$

$$(1.2 + 0{,}2\ S_{DS})\ D - \rho Ex + 0.3\ \rho Ey + 1.0\ L \tag{3-5}$$

$$(1.2 + 0{,}2\ S_{DS})\ D - \rho Ex - 0.3\ \rho Ey + 1.0\ L \tag{3-6}$$

$$(1.2 + 0{,}2\ S_{DS})\ D + 0.3\ \rho Ex + \rho Ey + 1.0\ L \tag{3-7}$$

$$(1.2 + 0{,}2\ S_{DS})\ D + 0.3\ \rho Ex - \rho Ey + 1.0\ L \tag{3-8}$$

$$(1.2 + 0{,}2\ S_{DS})\ D - 0.3\ \rho Ex + \rho Ey + 1.0\ L \tag{3-9}$$

$$(1.2 + 0{,}2\ S_{DS})\ D - 0.3\ \rho Ex - \rho Ey + 1.0\ L \tag{3-10}$$

$$(0.9 + 0{,}2\ S_{DS})\ D + \rho Ex + 0.3\ \rho Ey \tag{3-11}$$

$$(0.9 + 0{,}2\ S_{DS})\ D + \rho Ex - 0.3\ \rho Ey \tag{3-12}$$

$$(0.9 + 0{,}2\ S_{DS})\ D - \rho Ex + 0.3\ \rho Ey \tag{3-13}$$

$$(0.9 + 0{,}2\ S_{DS})\ D - \rho Ex - 0.3\ \rho Ey \tag{3-14}$$

$$(0.9 + 0{,}2\ S_{DS})\ D + 0.3\ \rho Ex + \rho Ey \tag{3-15}$$

$$(0.9 + 0{,}2\ S_{DS})\ D + 0.3\ \rho Ex - \rho Ey \tag{3-16}$$

$$(0.9 + 0{,}2\ S_{DS})\ D - 0.3\ \rho Ex + \rho Ey \tag{3-17}$$

$$(0.9 + 0{,}2\ S_{DS})\ D - 0.3\ \rho Ex - \rho Ey \tag{3-18}$$

3.2 Designing Earthquake Load

3.2.1 Risk category building structure

Risk category table defined by this clacification written based on SNI 1726:2012 procedures that author obtained from ASCE 7-10.

Table 3.1 Risk Category of Buildings and Other Structures

Use or Occupancy of Buildings and Structures	**Risk Category**
Buildings and other structures that represent a low risk to human life in the event of failure.	I
All buildings and other structures except those listed in Risk Categories I, III, and IV but required for: - Housing - Market - Factory Storage - Office Building - Apartment Building - Mega factory - Manufacture Facilities	II
Buildings and other structures, the failure of which could pose a substantial risk to human life. Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure. Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where their quantity exceeds a threshold quantity established by the authority having jurisdiction and is suffi cient to pose a threat to the public if released.	III
Buildings and other structures designated as essential facilities. Buildings and other structures, the failure of which could pose a substantial hazard to the community. Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of	IV

such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing suffi cient quantities of highly toxic substances where the quantity exceeds a threshold quantity established by the authority having jurisdiction to be dangerous to the public if released and is suffi cient to pose a threat to the public if released.a Buildings and other structures required to maintain the functionality of other Risk Category IV structures.	

Table 3.2 Earthquake Main Factor

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

3.2.2 Site Classification for Seismic Design

Site classification procedure defined on SNI 1726:2012 section 5.3 Table 3.3 using ASCE 7-10 Section 20.3 Site Class Definition to fit the language, the table shown below:

Table 3.3 Site Classification

Site Class	$\bar{v}_s$ **(m/second)**	$\bar{N}$ **atau** $\bar{N}_{ch}$	$\bar{s}_u$ (kPa)
SA (hard rock)	> 1500	N/A	N/A
SB (rock)	750 – 1500	N/A	N/A
SC (very dense and soft rock)	350 – 750	> 50	≥ 100
SD (stiff soil)	175 – 350	15 – 50	
SE (soft clay soil)	< 175	< 15	< 50
	Any profi le with more than 10 ft of soil having the following characteristics: —Plasticity index PI > 20, —Moisture content w ≥ 40%,		

	—Undrained shear strength < 25 kPa
SF (soils requiring site response analysis)	Any profile layers are following this one requirement or more: 1. Soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils. EXCEPTION: For structures having fundamental periods of vibration equal to or less than 0.5 s, site response analysis is not required to determine spectral accelerations for liquefiable soils. Rather, a site class is permitted to be determined in accordance with Section 20.3 and the corresponding values of Fa and Fv determined from Tables 11.4-1 and 11.4-2. 2. Peats or highly organic clays [H > (3 m)] of peat and/or highly organic clay where H = thickness of soil. 3. Very high plasticity clays [H > 25 ft (7.6 m) with PI > 75]. 4. Very thick soft/medium stiff clays [H > 120 ft (37 m)] with su < 1,000 psf (50 kPa).

3.2.3 Mapped Acceleration Parameter

The parameters S_s (short period of bedrock acceleration) and S_1 (period of bedrock acceleration at 1 second) SS and S1 shall be determined from the 0.2 and 1 s spectral response accelerations shown in section 14 with the probability 2% more than 50 years and defined by decimal due to gravity accelerations. If $S_1 \leq 0{,}04g$ and $S_s \leq 0{,}15g$ so the structure is permitted to be assigned to Seismic Design Category A and is only required to comply with Section 6.6 in SNI 1726:2012.

3.2.4 Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters Based on SNI 1726:2012

The MCER spectral response acceleration on the ground surface, needs seismic amplification at 0,2 s periods and 1 s period. Amplification factor included frequency short period (Fa). Parameter for short periods (SMS) and at 1 s (SM1), adjusted for Site Class effects, shall be determined by these Eqs:

$$S_{MS} = F_a S_S \tag{3-19}$$

$$S_{M1} = F_v S_1 \tag{3-20}$$

where

S_S = the mapped MCE_R spectral response acceleration parameter at short periods

S_1 = the mapped MCE_R spectral response acceleration parameter at a period of 1 s

Table 3.4 Site Coefficient, F_a

Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at Short Period.					
Site Class	Ss ≤ 0,25	Ss = 0,5	Ss = 0,75	Ss = 1,0	Ss ≥ 1,25
S_a	0,8	0,8	0,8	0,8	0,8
S_b	1,0	1,0	1,0	1,0	1,0
S_c	1,2	1,2	1,1	1,0	1,0
S_d	1,6	1,4	1,2	1,1	1,0
S_e	2,5	1,7	1,2	0,9	0,9
S_f	SS^D				

3.2.5 Design Spectral Acceleration Parameters

Table 3.5 Site Coefficient, F_y

Mapped Risk-Targeted Maximum Considered Earthquake (MCER) Spec- tral Response Acceleration Parameter at 1-s Period.					
Site Class	$S_1 \leq 0.25$	$S_1 = 0.5$	$S_1 = 0,75$	$S_1 = 1,0$	$S_1 \geq 1,25$
S_a	0,8	0,8	0,8	0,8	0,8
S_b	1,0	1,0	1,0	1,0	1,0
S_c	1,7	1,6	1,5	1,4	1,3
S_d	2,4	2,0	1,8	1,6	1,5
S_e	3,5	3,2	2,8	2,4	2,4
S_f	SS^D				

Design earthquake spectral response acceleration parameter at short period, S_{Ds}, and at 1 s period, S_{D1}, shall be determined from Eqs (3-21) and (3-22) based on SNI 1726;2012:

$$S_{DS} = \frac{2}{3} S_{MS} \quad (3\text{-}21)$$

$$S_{D1} = \frac{2}{3} S_{M1} \quad (3\text{-}22)$$

Where

S_{Ds} = the design spectral response acceleration parameter at short periods

S_{D1} = the design spectral response acceleration parameter at 1-s period

3.2.6 Seismic Design Category

According to S_{DS} and S_{D1} so seismic categories can be design by these table as shown below:

Table 3.6 Seismic Design Category Based on Short Period Response Acceleration Parameter

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

Table 3.7 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

Nilai S_{D1}	Keategori risiko	
	I atau II atau 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

To determine design coefficient and factors for seismic force-resisting system and reinforced concrete for the structure based on seismic design category S_{DS} and S_{D1}

Table 3.8 Structure type based on Seismic design category

Seismic Design Category	Seismic Force Resisting System
A, B	Frames System - Ordinary Moment Frames - Immediate Moment Frames - Special Moment Frames Wall System - Ordinary Structural Wall System - Special Structural Wall System
C	Frames System - Immediate Moment Frames - Special Moment Frames

	Wall System - Ordinary Structural Wall System - Special Structural Wall System
D, E, dan F	Frames System - Special Moment Frames Wall System - Special Structural Wall System

3.2.7 Approximate Fundamental Period

The approximate fundamental period (T_a), in s, shall be determined from the following equation:

$$T_a = C_t h_n^x \tag{3-23}$$

where h_n is the structural height and the coefficients C_t and x are determined from Table (dibawah). Alternatively, it is permitted to determine the approximate fundamental period (T_a), in s, from the following equation for structures not exceeding 12 stories above the base where the seismic force-resisting system consists.

Table 3.9 Values of Approximate Period Parameters Ct and x

Structure Type	C_t	**x**
Moment-resisting frame systems in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from defl ecting where subjected to seismic forces:		
Steel moment-resisting frames	0,0724[α]	0,8
Concrete moment-resisting frames	0,0466[α]	0,9
Steel eccentrically braced frames	0,0731[α]	0,75
Steel buckling-restrained braced frames	0,0731[α]	0,75
All other structural systems	0,0488[α]	0,75

Table 3.10 Coefficient for Upper Limit on Calculated Period

Design Response Spectrum Acceleration Parameter Period 1 Second, S_{D1}	Koefisien C_u
≥0,4	1,4
0,3	1,4
0,2	1,5
0,15	1,6
≤0,1	1,7

3.2.8 Calculation of Seismic Response Coefficient

The seismic response coefficient, Cs, shall be determined:

$$C_s = \frac{S_{DS}}{\frac{R}{I_e}} \quad (3\text{-}23)$$

where

S_{DS} = the design spectral response acceleration parameter in the short period range

R = the response modification factor

I_e = the importance factor

C_s shall not be more than

$$C_s = \frac{S_{DS}}{T\frac{R}{I_e}} \quad (3\text{-}24)$$

C_s shall not be less than

$$C_s = 0.044\ S_{DS}\ I_e \geq 0.01 \quad (3\text{-}25)$$

3.2.9 Seismic Base Shear

The seismic base shear, V, in a given direction shall be determined:

$$V = C_S\ W \quad (3\text{-}26)$$

where

Cs = the seismic response coefficient

W = the effective seismic weight

3.2.10 Number of Modes

An analysis shall be conducted to determine the natural modes of vibration for the structure. The analysis shall include a sufficient number of modes to obtain a combined modal mass participation of at least 90 percent of the actual mass in each of the orthogonal horizontal directions of response considered by the model.

3.3 Reinforcement Concrete Structure

3.3.1 Design Strength

Design Strength provided by a member, its connections to other members, and its cross sections, in terms of flexure, axial load, shear, and torsion, shall be taken as the nominal strength calculated in accordance with requirements and assumptions, multiplied by strength reduction factors ϕ based on SNI 2847-2013 section 9.3.

Table 3.11 Reduction factors for design strength

No	Information	ϕ
1	Tension-controlled sections	0,9
2	Members with spiral conforming reinforcement	0,75
3	Other reinforcement members	0,65
4	Shear and torsion	0,75
5	Bearing on concrete (except for post-tensioned anchorage zones and strut-and-tie models)	0.65
6	Post-tensioned anchorage zones	0,85

7	Strut-and-tie models, struts, ties, nodal zones, and bearing areas in such models.	0,75
8	From the end of the member to the end of the transfer length	0,75
9	From the end of the transfer length to the end of the development length, shall be permitted to be linearly increased from	0,75-0,9

3.3.2 Slab Design

3.3.2.1 One-way slab

Table 3.12 Minimum thickness for non-prestressed beam and one-way slab if deflection neglected

	Minimum thickness, *h*			
Structure Component	Simple span	One-end continues	Two-end continues	Cantilever
	Structure component non-bearing or not connected to partition or others element that may caused a big deflection			
One-way massive slab	*l*/20	*l*/24	*l*/28	*l*/10
Lateral one-way slab or beam	*l*/16	*l*/18,5	*l*/21	*l*/8

NOTE:

Span length is in mm.

The number that taken directly to the structure elements with normal concrete or reinforcement 420 MPa. For other condition, those number should be modified as follows:

a. For lightweight concrete structures with equilibrium density, w_c, between 1440 to 1840 kg / m^3, (a) For lightweight concrete structures with equilibrium density, w_c , between 1440 to 1840 kg / m^3, this value must be multiplied by (1.65-0,0003 w_c) but not less than 1.09.
b. for f_y other than 420 MPa, the number must be multiplied by (0.4+ f_y /700)

3.3.2.2 Two-way slab

Table 3.13 for minimum slab without interior beam.

Yield strength, f_y (Mpa)	Without thickening			With thickening		
	Exterior Panel		Interior Panel	Exterior Panel		Interior Panel
	Without beam edge	With beam edge		Without beam edge	With beam edge	
280	$l_n/20$	$l_n/36$	$l_n/36$	$l_n/36$	$l_n/40$	$l_n/40$
420	$l_n/30$	$l_n/33$	$l_n/33$	$l_n/33$	$l_n/36$	$l_n/36$
520	$l_n/28$	$l_n/31$	$l_n/31$	$l_n/31$	$l_n/34$	$l_n/34$

* For two-way elements, l_n is the net span length in the long direction, measured face to face footing on plates without beams and face to face beam or other supports in other cases.
† For f_y in between the number given in the table, the minimum thickness should be determined by linear interpolation.
‡ Panel drop
§ Slab with beams in between columns along the exterior edge.
The α_{fm} value for the edge beam must not be less than 0.8.

Provisions for slab with beam between two support in all edges, minimum thickness, h, based on SNI 2847-2013 section as follows:

1. For $\alpha_{fm} \leq 0,2$ should follow table above.

2. For α_{fm} $2 \geq 0{,}2$, ***h*** should be less than:

   (3-29)

$$h = \frac{\ln\left(0,8 + \frac{f_y}{1400}\right)}{36 + 5\beta\left(\alpha_{fm} - 0,2\right)}$$

   Should be more than 125 mm;

3. For $\alpha_{fm} > 2$, ***h*** should be more than:

   (3-30)

$$h = \frac{\ln\left(0,8 + \frac{f_y}{1400}\right)}{36 + 9\beta}$$

   Should be more than 90 mm

4. For some non-continue egde beam should has stiffness ratio α_f more than 0,8 or as alternative minimum thickness defined by Eqs. (2) or (3) should be increased minimum 10% for non-continue section.

3.3.3 Beam Design

3.3.3.1 Beam Dimension

Dimension beam defined by table

3.3.3.2 Longitudinal member reinforcement for beam

Minimum requirement for flexural component based on SNI 2847-2013 Section 10.5, should be more than:

$$A_{s(\min)} = \frac{0,25\sqrt{f'_c}}{f_y} b_w d$$

And should be more than:

$$As_{(\min)} = \frac{1,4 b_w d}{f_y}$$

3.3.3.3 Transversal reinforcement for beam

Based on SNI 2847-2013 section 21.5.3.2 the first reinforcement should be located 50 mm from the support face and should be less than:

a. d/4
b. 6d
c. 150 mm

Space for each reinforcement

$$s = \frac{A_s f_y d}{V_s}$$

3.3.4 Column Design

3.3.4.1 Column dimension

Based on SNI 2847-2013 section 21.6.1 requirements of this section apply to special moment frame members that form part of the seismic-force-resisting system and that resist a factored axial compressive force P_u under any load combination exceeding $A_g f'_c/10$. These frame members shall also satisfy the condition describe as follows:

a. The shortest cross-sectional dimension, measure on a straight line passing through the geometric centroid, shall not be less than 300 mm.
b. The ratio of the shortest cross-sectional dimension to the perpendicular dimension shall not be less than 0.4.

3.3.4.2 Slenderness Column

Based on SNI 2847-2013 section 10.10.1 slenderness effect shall be permitted to be neglected in the following cases:

a. For compression members not braced against sideway when:

$$\frac{k\ell_u}{r} \leq 22$$

b. For compression members braced against sideway when:

$$\frac{k\ell_u}{r} \leq 34 - 12(M_1 / M_2) \leq 40$$

3.3.4.3 Longitudinal reinforcement for column

Based on SNI 2847-2013 section 21.6.3.1 the area of longitudinal reinforcement, A_{st}, shall not be less than $0.01A_g$ or more than $0.06A_g$ where:

$A_{st} = \rho bh$

3.3.4.4 Minimum flexure strength of column

Based on SNI 2847-2013 section 21.6.2 the flexural strength of the columns shall satisfy the Eq. (-)

$$\sum M_{nc} \geq (1,2)\sum M_{nb}$$

Where:

ΣM_{nc} = Sum of nominal flexural strength of columns framing into the joint, evaluated at the faces of the joint. Column flexural strength shall be calculated for the factored axial forces considered, resulting in the lowest flexural strength.

ΣM_{nb} = sum of nominal flexural strength of the beams framing into the joint, evaluated at the faces of the joint. In T-beam construction, where the slab in tension under moments at the face of the joint, slab reinforcement within an effective slab width shall be assumed to contribute to M_{nb} if the slab reinforcement is developed at the critical section for flexure.

3.3.4.5 Transversal reinforcement for column

Based on SNI 2847-2013 section 21.6.4.1 transfer reinforcement shall not be less than the largest requirements described as follows:

a. The depth of the member at the joint face or at the section where flexural yielding is likely to occur;

b. One-sixth of the clear span of the member
c. 450 mm

Shall be provided over a length lo from each joint face and on both sides of any section when flexural yielding is likely occur as a result of inelastic lateral displacement of the frame.

3.3.5 Shear wall

Core wall is a structural wall that located in the centre of the building to resist the lateral force. Shear wall behave as a ductile when earthquake happening and shows plastic hinge on a wall. There is a boundary element or the limitation to keep the structure ducktail during the earthquake force. According to FEMA P-1051 section 10.5.3.3 boundary element shall be provided if compression stress exceeds 0,2f'_c the boundary element is required if:

$$\frac{P_u}{A_g} + \frac{M_u}{I} > 0,2f'_c$$

When the minimum thickness of the wall:

$$b < \frac{1}{8}h$$

3.4 Methodology

Design procedure:

1. Classifying building as follows:
a. Building specification of structure system, height of building, location and function.
b. Material specification for design and self-weight multiplying with the building dimension.

c. Earthquake load of Indonesian earthquake maps from BMKG *(Badan Meteorologi, Klimatologi, dan Geofisika)*

2. Preliminary design of structure elements

   Designing the dimension of each elements by the loads that each element carrying, according to SNI 2847:2013 requirements.

3. Structure modelling, the preliminary design of dimension assumption is designed by software ETABS 2017. This software helped the modelling to determining forces, moments and deflections that acting on the structure as a respond from the loads.
4. Designing a lateral load resisting structure according to SNI 1726:2012 that written in section 3.3.
5. Reinforcement design by envelope forces as the node elements of the load case combination with the minimum or maximum selected result component is searched and using moment 3-3 as a bending moment.

Building design performance analyse by response spectrum as the purpose of this final project.

Procedure diagram of redesigning PT Kino Indonesia office of 23 stories building:

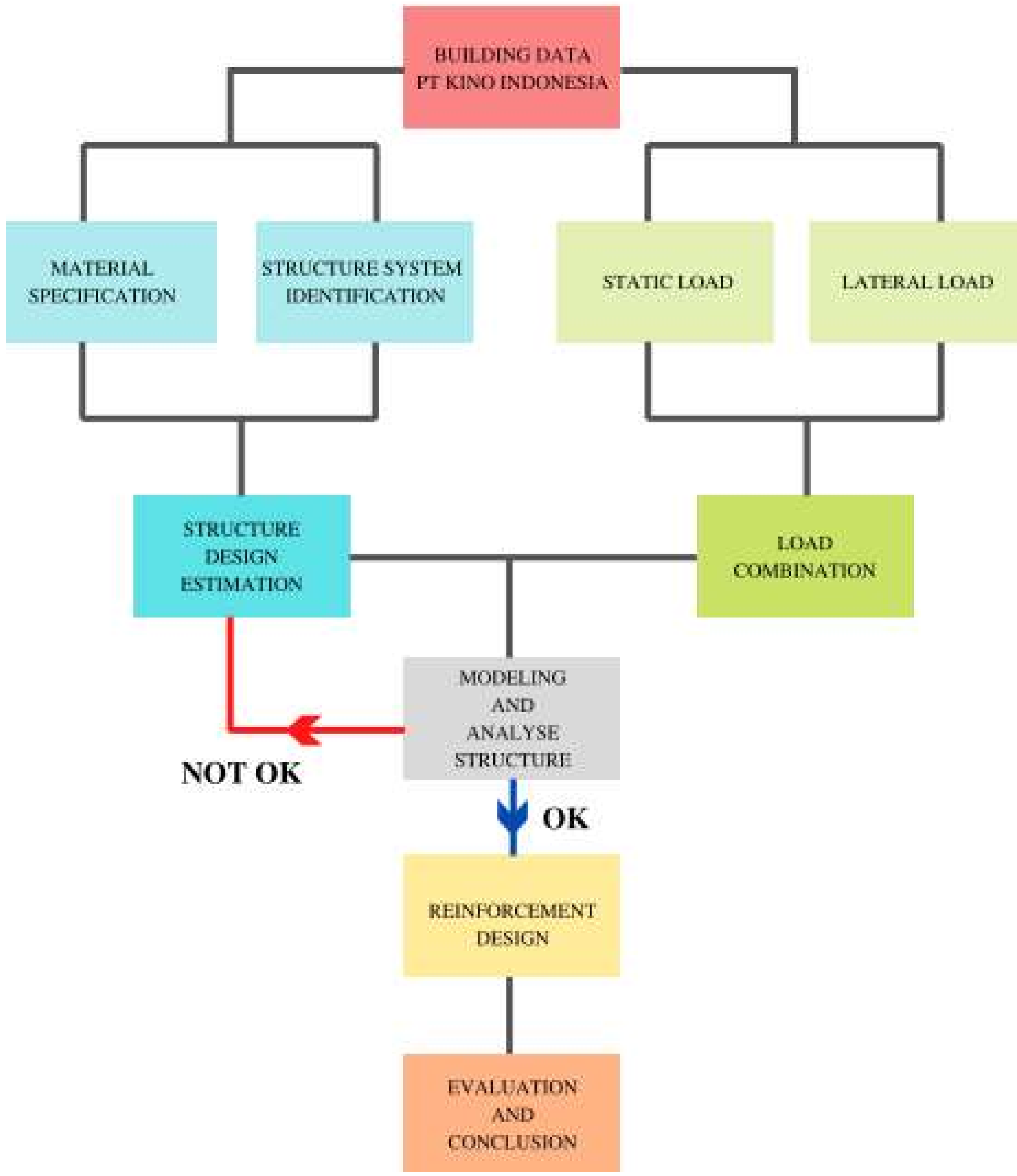

Fig 3.1. Flow Chart