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Analysis of SRPMK Implementation in PLN Office Building Design in Kupang City

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Abstract

Kupang City is located in a region with high seismic risk; Therefore, building design must comply with earthquake-resistant structural standards. This study aims to analyze the implementation of the Special Moment Resisting Frame System (SMRFS) in the structural planning of the PLN Office Building in Kupang City based onSNI 1726: 2019AndSNI 2847 : 2019.The research focuses on evaluating structural performance in terms of interstory drift, structural element capacity, application of the strong column–weak beam concept, and reinforcement detailing for seismic resistance. The study employed an engineering analysis approach using three-dimensional structural modeling with ETABS V20 software. The building analyzed is a three-story reinforced concrete office structure subjected to dead loads, live loads, and earthquake loads through equivalent static and response spectrum analyses. The results indicate that the maximum interstory drift reached 21,208 mm in the Y direction on the third floor, which is still below the allowable limit of 62 mm specified inSNI 1726: 2019. The designed structural dimensions consist of 40 × 60 cm main beams, 30 × 40 cm secondary beams, 60 × 60 cm columns, and 150 mm floor slabs. Shear capacity verification showed that the nominal shear strength exceeded the factored shear force, indicating adequate safety against shear failure. Furthermore, the strong column–weak beam requirement was satisfied because the column moment capacity was greater than that of the beams. Therefore, the implementation of SMRFS provides adequate strength, stiffness, stability, and ductility for resisting design earthquake loads.

Keywords Special Moment Resisting Frame System (SMRFS), Earthquake-Resistant Structure, Interstory Drift, Strong Column–Weak Beam, Etabs

INTRODUCTION

Kupang City, the capital of East Nusa Tenggara Province, is experiencing rapid infrastructure development, particularly in the construction of office buildings, public facilities, and high-rise buildings. This increased development demands structural planning that is not only economically efficient but also safe from potential disasters, particularly earthquakes.SNI 1726:2019Kupang City is known to be an earthquake-prone area with a high seismic risk. This is indicated by the short-period response spectral acceleration (Ss) parameter value, which ranges from 1.2 to 1.4 g. This condition requires that all building structural designs adhere to earthquake resistance principles in accordance with applicable standards.

In earthquake-resistant structural design, selecting the right structural system is crucial for determining a building's performance. One system recommended for areas with high seismicity is the Special Moment Resisting Frame System (SRPMK). This system is designed to have a high degree of ductility, allowing it to withstand significant inelastic deformation without sudden collapse.(Manga & Tanijaya, 2024AndManurung et al., 2025). The SRPMK principle refers to the strong column–weak beam concept, where the column moment capacity must be greater than the beam by fulfilling the ratio $\Sigma Mnc \geq 1.2\ \Sigma Mnb$ according toSNI 2847: 2019so that the collapse mechanism that occurs is ductile and controlled. With this principle, SRPMK is able to withstand the inelastic response cycle when receiving the planned earthquake load.(Di et al., 2024AndWildan & Lestyowati, 2025).

Several previous studies have shown that the application of SRPMK is able to improve the performance of structures in resisting earthquake loads through more controlled energy dissipation mechanisms and plastic hinge distribution.(Basyir et al., 2022 andWardi et al., 2022)However, to date, there has not been much research that specifically evaluates the implementation of SRPMK in office buildings in the Kupang area by considering local seismic parameters and verifying the strong column–weak beam principle based on the results of numerical analysis.

The PLN Kupang City Office Building, as a critical office building, requires structural design that meets high earthquake safety criteria. Therefore, a comprehensive analysis of the implementation of the SRPMK in the building is required, including aspects of strength, ductility, and overall structural performance. Structural modeling using ETABS V20 software allows for a more accurate analysis of the structural response to a combination of applied loads, including earthquake loads.

Based on the description, this study aims to analyze the implementation of the Special Moment Resisting Frame System (SRPMK) in the planning of the PLN Kupang City Office Building based on the provisionsSNI 1726:2019,SNI 2847:2019,SNI 1727:2020and evaluate the structural performance including internal forces, story drift, and the application of the strong column–weak beam principle. This research contributes in the form of a comprehensive evaluation of the structural performance based on SRPMK in office buildings in high seismic areas using a numerical analysis approach using ETABS. The results of this study are expected to serve as a reference in the structural planning of high-rise buildings in earthquake-prone areas and support the implementation of national standards in earthquake-resistant building design. A Special Moment Resisting Frame System (SRPMK) is a structural system designed to provide high ductility, namely the ability of a structure to undergo significant plastic deformation without collapsing. This system is designed with very strong and rigid beam-column connections, which allows the building to withstand large earthquake forces.(Manga & Tanijaya, 2024).

The strong column–weak beam principle is a basic concept in SRPMK planning that aims to avoid undesirable collapse mechanisms. In this principle, the column moment capacity must be greater than the beam moment capacity at each beam–column connection. This concept ensures that structural elements such as columns remain strong and do not undergo plasticification, while other elements, such as beams, are allowed to experience plastification to dissipate earthquake energy. According toSNI 2847: 2019The ratio of column to beam strength must meet certain requirements so that plasticity occurs first in the beam. This way, structural damage can be controlled and occurs gradually (ductile failure), thus giving occupants time to evacuate.(Manga & Tanijaya, 2024).

Story drift is an important parameter in evaluating the performance of a structure against earthquake loads. According toSNI 1726:2019, inter-story drift is defined as the difference in lateral displacement between two consecutive floors due to the influence of earthquake loads. The standard also sets a maximum limit for inter-story drift to prevent structural and non-structural damage to buildings. Story drift values exceeding the permissible limits can cause damage to non-structural elements such as infill walls, partitions, and other architectural components, and can even affect the overall stability of the structure. Furthermore, excessive drift can also reduce the comfort level of building occupants during an earthquake.(Chopra, 2020).

2. METHOD

This research is an engineering study that aims to analyze the implementation of the Special Moment Resisting Frame System (SRPMK) in the reinforced concrete structure planning of the PLN Kupang Office Building located at Jln. Palapa No. 27, Oebobo Village, Oebobo District, Kupang City, East Nusa Tenggara Province in an area with high earthquake risk. The structural planning refers to the provisions of SNI 2847:2019 and SNI 1726:2019.

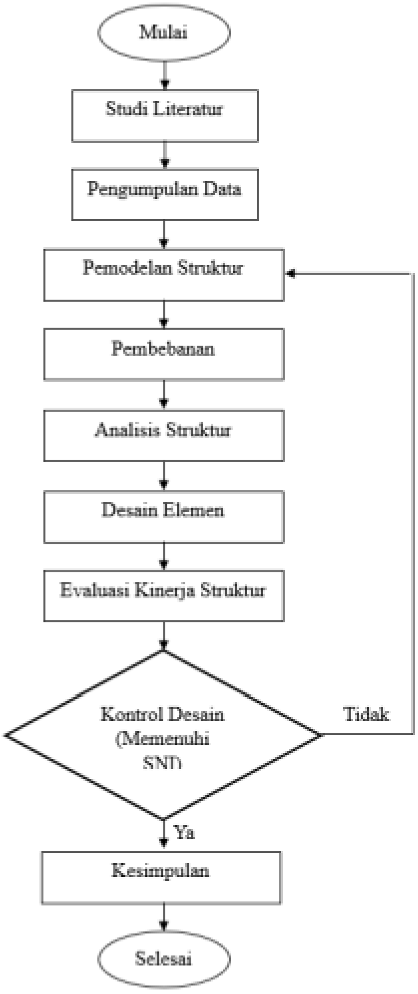

Figure 1. Research Flowchart

The research method begins with a literature study to understand the SRPMK concept and earthquake-resistant design provisions, then continues with the determination of planning parameters including building function, number of floors, land site classification, and seismic parameters based on the Kupang location. The structure is modeled three-dimensionally using ETABS V20 software to obtain the structural response due to dead loads, live loads, and earthquake loads through equivalent static analysis and spectrum response. The results of the analysis in the form of internal forces and deformations are used as the basis for evaluating the main structural elements (beams and columns) according to the SRPMK concept, including strong column–weak beam inspection and reinforcement detailing. The final stage is an evaluation of the structural performance in terms of strength, stiffness, and ductility aspects based on standard limits. If the analysis results do not meet the requirements, planning iterations are carried out until a safe and compliant structural design is obtained. This research is limited to the analysis of the main structure and does not include architectural aspects and the mechanical and electrical systems of the building.

Structural planning data and material specifications

1. Building data
   Building function:Office (PLN Kupang City)
   Location :Jln. Palapa No. 27, Oebobo Village, Oebobo District, Kupang City, East Nusa Tenggara Province
   Number of floors : 3 (three)
   Height of 1st floor : 4 m
   2nd floor height : 4 m
   3rd floor height : 4 m

2. Material data
   Concrete quality : 25 MPa
   Threaded steel quality (fy)) : 420 MPa
   Plain steel grade (fy) : 280 MPa
   Modulus of elasticity of steel : 200,000 MPa
   Modulus of elasticity of concrete: 23500 MPa

***Preliminary*Dimensions of Structural Elements**

1. Beam
   B1 (main beam) : 40/60 cm
   BA-1 (children's block): 30/40 cm
   RB-1 (ring beam) : 40/60 cm
2. Column
   K1 (structural column): 60/60 cm
   K2 (practical column): 15/15 cm
3. Floor plates
   PL (conventional floor plate): 150 mm (thickness)
4. Sloof
   Sloof (SL-1): 30/40 cm
   Sloof (SL-2): 20/30 cm

Loading

RegulationSNI 1727: 2020is a reference used in determining the load for building planning andSNI 1726: 2019as a reference in the procedures for planning resilience for building structures.

1. Dead load
   The dead load in this study consists of the structural dead load and additional dead load (non-structural) that acts permanently on the building.
   Dead load of structure (volume weight of reinforced concrete) = 2400 kg/m3

Dead load of floor = 130 kg/m2
Dead load of wall = 2.79 KN/m
Dead load of roof floor = 93 kg/m2
Roof Load (Light Steel Frame + Cover) = 3.23 KN/m

2. burden of life
The live load in this design refers to the function of the space in an office building and is expressed as a uniform load acting on the floor slab. The live load values used are presented in the following table.

Table 1. Live load

Distributed live load of building structures	
Occupancy or use	**Evenly (KN/m2)**
Rg. Office, working space, etc	2.40
Rg. Fitness/Gym	3
Men's/women's toilets, etc.	1.92

3. Earthquake load
The earthquake loads in this study were analyzed using ETABS software through three-dimensional structural modeling. The analysis was conducted using two approaches: equivalent static analysis and dynamic analysis (response spectrum). Static analysis was used to obtain seismic forces based on seismic parameters and building characteristics, while dynamic analysis was used to obtain a more accurate structural response by considering the dynamic properties of the building. The results of this analysis were used as a basis for determining the forces in structural elements according to the SRPMK planning provisions.
Ss = 1.0497
S1 = 0.375
Design category D

Load Combination

Combination 1 = 1.4D + 1.4SID
Combination 2 = 1.2D + 1.2 SID + 1.6L
Combination 3 = 1.24D + 1.24SID + 0.3L + 0.39SpecX + 1.3SpecY
Combination 4 = 1.22D + 1.22SID + 0.3L - 0.39SpecX + 1.3SpecY
Combination 5 = 1.18D + 1.18SID + 0.3L + 0.39SpecX - 1.3SpecY
Combination 6 = 1.16D + 1.16SID + 0.3L -0.39SpecX - 1.3SpecY

Combination 7 = 1.24D + 1.24SID + 0.3L + 1.3SpecX + 0.39SpecY
Combination 8 = 1.18D + 1.18SID + 0.3L – 1.3SpecX + 0.39SpecY
Combination 9 = 1.22D + 1.22SID + 0.3L + 1.3SpecX – 0.39SpecY
Combination 10 = 1.16D + 1.16SID + 0.3L – 1.3SpecX – 0.39SpecY
Combination 11 = 0.86D + 0.86SID + 0.39SpecX + 1.3SpecY
Combination 12 = 0.88D + 0.88SID - 0.39SpecX + 1.3SpecY
Combination 13 = 0.92D + 0.92SID + 0.39SpecX - 1.3SpecY
Combination 14 = 0.94D + 0.94SID - 0.39SpecX - 1.3SpecY
Combination 15 = 0.86D + 0.86SID + 1.3SpecX + 0.39SpecY
Combination 16 = 0.92D + 0.92SID +-1.3SpecX + 0.39SpecY
Combination 17 = 0.88D + 0.88SID + 1.3SpecX - 0.39SpecY
Combination 18 = 0.94D + 0.94SID - 1.3SpecX - 0.39SpecY

3. RESULTS AND DISCUSSION

Structural Modeling and Structural Analysis

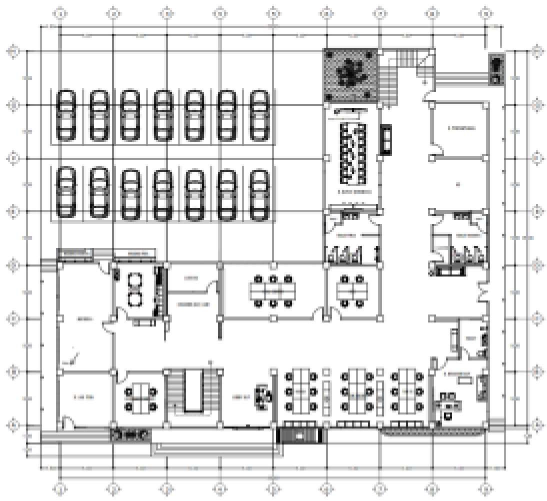

Figure 2. Floor Plan 1

Figure 3. Floor Plan 2

Figure 4. Floor Plan 3

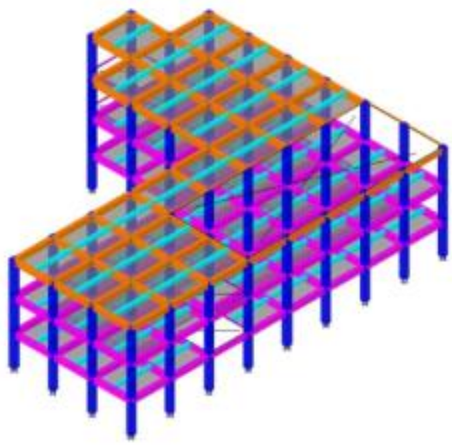

Figure 5. 3D View of Structural Modeling

Interlevel Deviation

Based on the results of the structural analysis using ETABS, the interstory drift value was obtained as presented in Table 2. The drift was calculated as the difference in lateral displacement between floors due to earthquake loads, which was then multiplied by a magnification factor to obtain the inelastic drift according to the provisions.SNI 1726: 2019.

Table 2. Inter-Level Deviation

Story	Displacement		Elastic Drift		h	Inelastic Drift		Drift Limit	Check
	δe_x	δe_y	δe_x	δe_y		Δ_x	Δ_y		
	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	
3	6,200	6,580	3,600	3,856	4000	19,800	21,208	62	OK
2	2,600	2,724	2,600	2,724	4000	14,300	14,982	62	OK

The results show that on the 3rd floor, the inelastic deviations obtained were Δx = 19,800 mm and Δy = 21,208 mm, while on the 2nd floor they were Δx = 14,300 mm and Δy = 14,982 mm. The largest deviation value occurred in the Y direction on the 3rd floor, which was 21,208 mm.

When compared with the permissible deviation limits according to SNI 1726: 2019namely:

$\Delta_a = 0{,}020\ h$

$\rho = 1{,}3$

$$\Delta_{max} = \frac{\Delta_a}{\rho} = \frac{0{,}020\ x\ 4000}{1{,}3} = 62\ mm$$

Therefore, all deviation values that occur are still below the permitted limit (≤ 62 mm). Thus, the structure of the PLN Kupang City Office building meets the performance requirements for inter-story deviations.

Distributionally, it can be seen that the displacement increases with increasing floor height, with the upper floor (floor 3) experiencing greater displacement than the floors below. This is a common behavior in multi-story building structures due to the accumulation of lateral deformation from the bottom up.

Thus, it can be concluded that the implementation of SRPMK in this building has succeeded in providing safe structural performance and fulfilling the requirements.SNI 1726: 2019in terms of controlling inter-story drift, while also demonstrating the structure's ability to respond to earthquake loads in a ductile and controlled manner.

Column Planning

The planned column has dimensions of 60 cm x 60 cm, concrete cover (s) = 40 mm, and effective height (d) = 542 mm. The diameter of the main reinforcement = 16 mm and the stirrup reinforcement = 10 mm. In accordance withSNI 2847: 2019Article 18.7.4.1 regarding the column longitudinal reinforcement ratio is 1% < 1.12% < 6%, with 20D16 reinforcement (As = 4021.24 mm2).

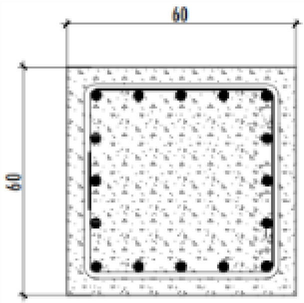

Figure 6. 60x60 column

The results of the analysis using ETABS, obtained internal forces in the column are presented in the table below.

Table 3. Internal Forces in 60x60 Columns

Condition	**P** (kN)	**Mx** (kNm)	**My** (kNm)	**Your** (kNm)
P max	914.67	-183,785	92,754	205,865
P min	18,304	-168,072	46,716	174,444
Mx max	77,247	336,144	59,597	314,386
Mx min	79,128	-336,144	56,335	340,832
My max	41,279	-44,277	336,144	339,048
My min	70,299	69,685	-336,144	343,291

Longitudinal reinforcement of the column, the Mn-Pn interaction diagram is calculated manually and the column interaction diagram is obtained as shown in the figure below.

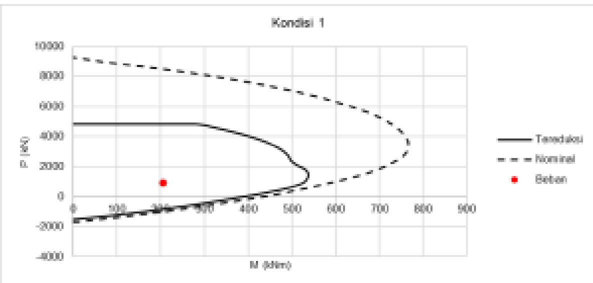

Figure 7. Interaction diagram of condition column 1

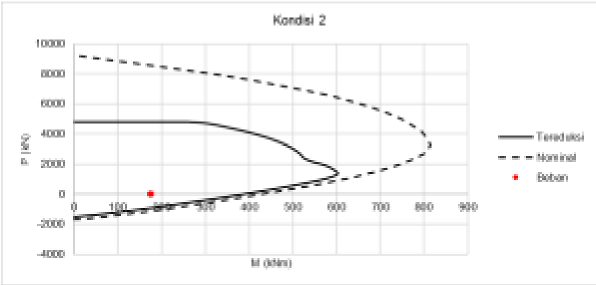

Figure 8. Interaction diagram of condition column 2

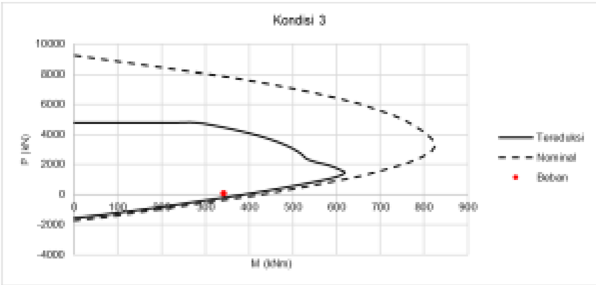

Figure 9. Interaction diagram of condition column 3

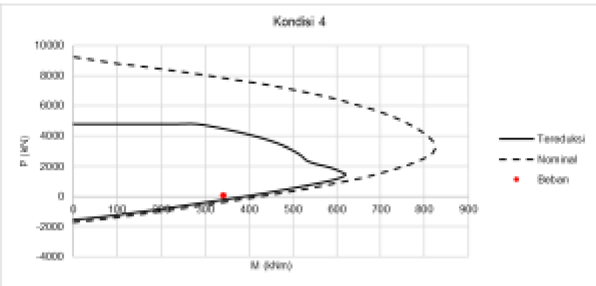

Figure 10. Interaction diagram of condition column 4

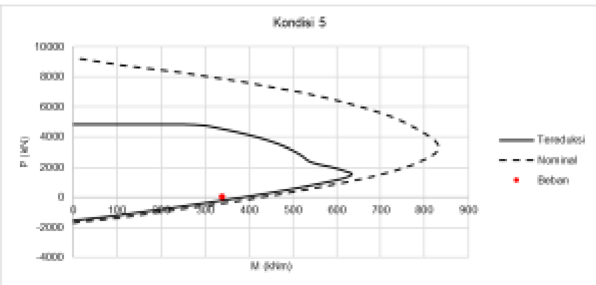

Figure 11. Interaction diagram of condition column 5

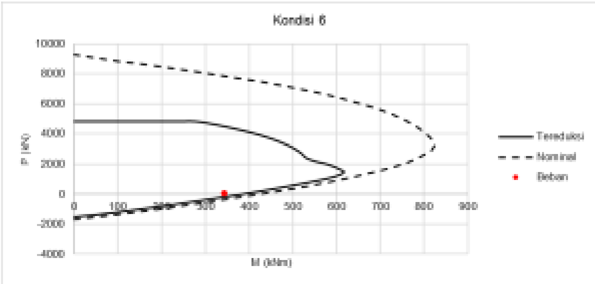

Figure 12. Interaction diagram of condition column 6

Based on the results of the reinforcement calculations in the lo area and outside the lo area, the column shear obtained is D10-60 mm and D10-80 mm respectively.

1) Check the conditions for strong columns – weak beams
   According to Capacity Design, the flexural strength of the column must meetSNI 2847: 2019Article 18.7.3.2, namely ΣMnc > 1.2 ΣMnb.
   X column moment
   ΣMnc > ΣMnb = 1095.6 KNm > 512.928 KNm (conditions met)
   Column Y Moment
   ΣMnc > ΣMnb = 552.9 KNm > 512.928 KNm (conditions met)
2) Beam-column relationship
   a) Support
      The working shear force (Vu) = 402821.47 N. The concrete shear capacity (Vc) = 277424 N, while the reinforcing steel shear capacity (Vs) = 89390.2 N.
      Thus, the nominal capacity of the column is obtained as Vn = ϕ (Vc + s) = 878523 N.
      Value Vn = 878523 N > Vu = 402821.47 N (SAFE).
   b) Field
      The working shear force (Vu) = 192897 N. The shear capacity of concrete (Vc) = 277424 N, while the shear capacity of reinforcing steel (Vs) = 446970 N.
      Thus, the nominal capacity of the column is obtained as Vn = ϕ (Vc + s) = 543295 N.
      Value Vn = 543295 N > Vu = 192897 N (SAFE).

The column's shear capacity, which is much greater than the applied shear force, indicates that the column element has a high level of safety against shear failure. This condition indicates that the likely failure mechanism is flexural failure of the beam, in accordance with the strong column–weak beam concept expected in earthquake-resistant design.

Beam Design

The planned main beam dimensions are 40 cm x 60 cm, concrete cover (s) = 40 mm, main reinforcement = 16 mm and stirrup reinforcement = 10 mm. The results of the analysis using ETABS, obtained internal forces in the beam are presented in the table below.

Table 4. Internal Forces in a 40x60 Beam

Inner style	Mark	Unit
Mu tump (-)	-151,669	KNm
Mu tump (+)	94,842	KNm
Mu lap (-)	-27,749	KNm
Mu lap (+)	48,689	KNm
Vu tump	164,953	KN
Vu lap	133,759	KN
Vg tump	82,208	KN
Pu	0	KN

1) Required reinforcement
   Support area reinforcement
   a) Tensile area reinforcement
      Number of reinforcement layers 1 = 4
      Number of reinforcement layers 2 = 0
      Astulangan = As1 + As2
      = 804,248 + 0
      = 804.248 mm^2
      Asmin $= \frac{\sqrt{fc}}{4fy} b.d$
      = 645.2 mm2
      Asmin $= \frac{1,4}{fy} b.d$
      = 722.7 mm^2
      Asmin = 722.7 mm2 < Astulangan = 804.248 mm2 (OK)
   b) As the compression area reinforcement
      Number of reinforcement layers 1 = 4
      Number of reinforcement layers 2 = 2
      Astulangan = As1 + As2
      = 804,248 + 403,124
      = 1206.4 mm^2
      Asmin $= \frac{\sqrt{fc}}{4fy} b.d$

= 645.2 mm2

Asmin $=\frac{1,4}{fy} b.d$

= 722.7 mm²

Asmin = 722.7 mm2 < Reinforcement = 1206.4 mm2 (OK)

Field area reinforcement

a) Tensile area reinforcement

Number of reinforcement layers 1 = 4

Number of reinforcement layers 2 = 2

Astulangan = As1 + As2

= 804,248 + 403,124

= 1206.4 mm²

Asmin $=\frac{\sqrt{fc}}{4fy} b.d$

= 645.2 mm2

Asmin $=\frac{1,4}{fy} b.d$

= 722.7 mm²

Asmin = 722.7 mm2 < Reinforcement = 1206.4 mm2 (OK)

b) As the compression area reinforcement

Number of reinforcement layers 1 = 4

Number of reinforcement layers 2 = 0

Astulangan = As1 + As2

= 804,248 + 0

= 804.248 mm²

Asmin $=\frac{\sqrt{fc}}{4fy} b.d$

= 645.2 mm2

Asmin $=\frac{1,4}{fy} b.d$

= 722.7 mm²

Asmin = 722.7 mm2 < Astulangan = 804.248 mm2 (OK)

2) Check the factored nominal moment against the ultimate moment

Support area reinforcement

a) Tensile reinforcement

ϕMn = 158,731 KNm > Mu = 94,842 KNm (satisfied)

b) Compression area reinforcement

ϕMn = 225,965 KNm > Mu = 151,669 KNm (satisfied)

Field area reinforcement

c) Tensile reinforcement

ϕMn = 225,965 KNm > Mu = 48,689 KNm (satisfied)

d) Compression area reinforcement

ϕMn = 158,731 KNm > Mu = 27,749 KNm (satisfied)

3) Planned shear reinforcement

a) Design shear force

Ve = Vg + Vpr

= 82,208 + 119,882

= 202.09 KN

b) The concrete shear strength Vc used in the design

SNI 2847: 2019Article 18.6.5.2 states that for shear reinforcement along the lo area, the concrete shear strength Vc is taken as zero if the following two requirements are met:

$V_{pr} = 119{,}882 \geq \frac{V_e}{2} = 101{,}045$(fulfil)

$P_u = 0 < \frac{A_g x f_c}{20} = 300$(fulfil)

c) Required shear strength of reinforcement, Vs

Ve = 202.09 KN; ϕ = 0.75; Vc = 0 KN

Vsmin = = 446,970 KN$\frac{A_v x f_y x d}{s}$

Check against Vsmaks

Vs = 446,970 KN < = 715,440 KN (OK)$0{,}66\sqrt{f_c} x b x d$

For the support area

Shear reinforcement with a diameter of 10 mm (2 feet) is installed. Av = 157.08 mm2. Actual = 80 mm and Smax = 96 mm, because Actual < Smax, the shear reinforcement distance used in the lo area is 80 mm.

For field area

Shear reinforcement with a diameter of 10 mm (2 feet) is installed. Av = 157.08 mm2. Actual = 150 mm and Smax = 271 mm, because Actual < Smax, the shear reinforcement distance used in the lo area is 150 mm.

Discussion

Based on the results of the structural analysis using ETABS, the implementation of the Special Moment Resisting Frame System (SRPMK) in the PLN Kupang City Office Building shows structural performance that meets earthquake resistance requirements based onSNI 1726: 2019AndSNI 2847 : 2019.This is reviewed from the perspective of inter-story deviation control, structural element capacity, strong column–weak beam concept, and earthquake-resistant reinforcement detailing. The results of the inter-story drift analysis show that the maximum drift value occurred on the 3rd floor in the Y direction, at 21.208 mm. This value is still below the permissible drift limit of 62 mm as stipulated in the regulations.SNI 1726 : 2019.This condition indicates that the structure has sufficient lateral stiffness to withstand the effects of earthquake forces. The results of this study are in line with research conductedDewi et al., 2023which states that the application of SRPMK can control inter-story drifts, keeping them within the safe limits required by Indonesian earthquake standards. Furthermore, the use of SRPMK also provides better energy dissipation capabilities than conventional moment-resisting frame systems.

In the column elements, the longitudinal reinforcement ratio of 1.12% has met the requirements.SNI 2847: 2019which is in the range of 1%–6%. The PM interaction diagram shows that all combinations of axial forces and moments remain within the safe zone of cross-sectional capacity. This indicates that the column has sufficient capacity to withstand the combination of gravity and lateral loads due to the earthquake. Examination of the strong column–weak beam concept shows that the column moment capacity is greater than the beam moment capacity in both principal directions of the structure. This condition fulfills the capacity design principle in SRPMK, where the collapse mechanism is expected to occur first in the beam before the column fails.

In the beam-column connection, the nominal shear capacity obtained is greater than the factored shear force acting at both the support and field areas. This indicates that the structural element is safe from shear failure. The use of close-set stirrups in the plastic hinge area also increases the structure's ductility and energy dissipation capacity during strong earthquakes. Overall, the implementation of the SRPMK at the PLN Kupang City Office Building successfully produced a structure that meets the strength, stiffness, and ductility requirements of Indonesian earthquake-resistant design standards. Thus, the structure is expected to provide safe and stable performance under planned earthquake loads while minimizing the risk of structural collapse.

4. CONCLUSION

Based on the design results, several conclusions can be drawn regarding the structural performance of the PLN Kupang City Office Building. The structural analysis conducted using ETABS indicates that the building, which applies the Special Moment Resisting Frame System (SRPMK), has fulfilled the earthquake-resistant design requirements in accordance with SNI 1726:2019 and SNI 2847:2019. This demonstrates that the structural system is capable of resisting seismic forces safely and effectively. The analysis results also show that the maximum inter-story drift occurred on the third floor in the Y direction with a value of 21.208 mm. This value is still far below the allowable limit of 62 mm, indicating that the structure is safe against excessive lateral deformation caused by earthquake loads. Therefore, the building satisfies the serviceability and stability requirements under seismic loading conditions. From the structural design process, the dimensions of the structural elements were determined as follows: the main beams have dimensions of 40 × 60 cm, the secondary beams measure 30 × 40 cm, the columns are designed with dimensions of 60 × 60 cm, and the slab thickness is 150 mm. These dimensions were selected to ensure adequate strength, stiffness, and structural performance under both gravity and lateral loads. Furthermore, the shear capacity evaluation on both beams and columns indicates that the nominal shear capacity (Vn) is greater than the factored shear force (Vu). This confirms that all structural elements are safe against shear failure. In addition, the strong column–weak beam concept has been successfully implemented, where the moment capacity of the columns exceeds that of the beams in both structural directions. As a result, the intended ductile failure mechanism can be achieved, allowing the structure to dissipate earthquake energy effectively and minimizing the risk of sudden collapse during strong seismic events.

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