

Modified Superstructure Design of the Religious Court Office Building in Penajam Paser Utara Regency, East Kalimantan Province

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Abstract

Vertical extension is increasingly adopted to meet the growing space demand of public buildings, but it increases gravity and seismic demands and requires structural redesign in accordance with current standards. This study redesigns the superstructure of the Religious Court Office Building of Penajam, located in the development area of Indonesia's new capital city (IKN), from two to three storeys. The design follows SNI 1726:2019, SNI 2847:2019, SNI 1727:2020, and SNI 1729:2020. The structure was modelled as a three-dimensional reinforced concrete special moment frame in SAP2000 v24 and analysed using the equivalent lateral force procedure. The results show that the building is classified as Seismic Design Category B, with a design base shear of 431.3 kN, or 1.6% of the effective seismic weight. The 120 mm slabs require Ø10-100 reinforcement at supports and Ø10-200 at midspan. The main beam (350×700 mm) requires 7D19 at the supports, and the main column (500×500 mm) requires 12D22, with a maximum flexural demand-to-capacity ratio of 0.98. Most members are governed by minimum reinforcement and detailing provisions rather than by seismic strength demand, as indicated by the governing role of maximum stirrup spacing and minimum torsional reinforcement in all beams. These findings indicate that in low-seismicity regions such as IKN, the choice of seismic force-resisting system affects detailing and constructability more than member dimensions.

Keywords: Vertical Extension; Low Seismicity; Special Moment Frame; Reinforced Concrete Design; SNI 1726:2019



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INTRODUCTION

The demand for public service buildings continues to grow with population growth and increasing service volumes, while available land in urban areas is becoming increasingly limited. Vertical extension, the addition of storeys to an existing building, has therefore become a widely adopted solution. Compared with demolition and reconstruction, vertical extension is regarded as a more sustainable option because it reduces cost, environmental pollution, and construction waste (Shala & Bleiziffer, 2024). However, adding storeys also increases the gravity and seismic demands that the structure must resist. Bahrami et al., (2022) modelled the vertical extension of a reinforced concrete building and found that several load-bearing elements required strengthening after the storey was added. Shala & Bleiziffer (2024) further showed that the connection between the existing structure and the new storeys is commonly assumed to be rigid in structural analysis, even though its actual behaviour may differ. These findings indicate that a change in the number of storeys requires a comprehensive structural redesign, not merely the placement of new elements on top of the existing structure.

This requirement is particularly important in Indonesia, where the national seismic design code has been updated to SNI 1726:2019. The update has had a substantial effect on design seismic forces. For example, a comparative study of a reinforced concrete building in Malang reported differences in base shear of 47% in the Y direction and 69% in the X direction between SNI 1726:2012 and SNI 1726:2019 (Sarya et al., 2021). Beyond the magnitude of

seismic forces, the quality of structural detailing is equally decisive for building safety. Field observations after the 2023 Kahramanmaraş earthquakes showed that damage to reinforced concrete buildings was largely attributable to design-stage errors, including strong beam weak column mechanisms, short columns, soft storeys, and insufficient transverse reinforcement (Altunışık et al., 2023). Seismic design therefore cannot rely solely on member strength checks; it must also satisfy capacity design principles and the ductile detailing requirements associated with the selected structural system.

The Religious Court Office Building of Penajam is located in Penajam Paser Utara Regency, East Kalimantan, which lies within the development area of Indonesia's new capital city, Ibu Kota Nusantara (IKN). Large-scale construction of infrastructure and government buildings is expected in this region. In terms of seismicity, the area is relatively safe. A probabilistic seismic hazard analysis (PSHA) of the new capital region for a 2,500-year return period yielded bedrock peak ground accelerations of approximately 0.2-0.25 g, and the hazard index was classified as low according to the national disaster management agency (Mountainshia et al., 2022). Nevertheless, Khalqillah et al., (2025) showed that although Kalimantan generally exhibits lower seismic hazard, East Kalimantan records localized high intensities associated with the Meratus and Mangkalihat faults. Consequently, the relevant design question in this region is not whether seismic loads should be considered, but which structural system and level of detailing offer an appropriate balance between safety and efficiency. A study in East Kalimantan comparing intermediate and special moment frames found identical beam, column, and slab dimensions for both systems; the difference lay in the closer spacing of transverse reinforcement required for the special moment frame owing to its stricter ductility provisions (Ramadhani et al., 2026).

Several studies in Indonesia have examined vertical extensions of reinforced concrete buildings, with contrasting outcomes. Subagio et al., (2021) evaluated the addition of three storeys and a rooftop to a lecture building at the Faculty of Engineering, Universitas Jember, using three-dimensional modelling. The model retained the existing member dimensions and added new storeys based on preliminary design. The extended building was found to be safe, with a maximum inter-storey drift of 10.2 mm, below the allowable limit of 31.5 mm. In contrast, Kurniawan et al., (2024) investigated a two-storey lecture building in Bukittinggi planned for extension to three storeys. They found that the column demands fell outside the interaction diagram, and pushover analysis indicated that the storey could not be added without strengthening the structural members and evaluating the foundation capacity. Internationally, Bahrami et al., (2022) compared a reinforced concrete building in Sweden before and after vertical extension and concluded that the extension was feasible once certain load-bearing elements had been strengthened. Together, these studies demonstrate that the feasibility of vertical extension depends strongly on the capacity of the original structure and the seismicity of the site. The results of one case therefore cannot be generalized to other buildings, and each building requires a site-specific analysis.

Nevertheless, these studies have several limitations with respect to the context of the present research. First, vertical extension studies in Indonesia have mostly addressed educational buildings on Java and Sumatra, which are regions of moderate to high seismicity. Second, they have focused on assessing the capacity of existing structures rather than on a complete redesign in accordance with the current codes. The East Kalimantan study by Ramadhani et al., (2026) compared structural systems for a new building rather than for a building undergoing a change in the number of storeys. To the best of the authors' knowledge, no study has specifically addressed the redesign of the superstructure of a government building, particularly a judicial building, extended from two to three storeys in the low-

seismicity IKN region. Such a study should verify not only member strength but also inter-storey drift, structural stability, column slenderness, and the detailing requirements of the selected moment-resisting frame system. Judicial buildings also contain specific occupancies, such as courtrooms and archive rooms, that require live loads different from those of ordinary office buildings. To address this gap, the present study seeks to answer the following research question: how should the superstructure of the Religious Court Office Building of Penajam be redesigned to safely accommodate its extension from two to three storeys in accordance with the current Indonesian standards in a low-seismicity region?

To answer this question, this study redesigns the superstructure of the Religious Court Office Building of Penajam in accordance with SNI 1726:2019, SNI 2847:2019, SNI 1727:2020, and SNI 1729:2020, using SAP2000 v24 for structural modelling and analysis. The design covers the determination of seismic parameters and the Seismic Design Category, as well as the design of the steel roof truss, slabs, stairs, beams, and columns. The main contribution of this study is a complete record of a structural redesign for a low-rise judicial building undergoing vertical extension in a low-seismicity region. The results are expected to serve as a reference for the design of government buildings in the rapidly developing IKN region.

RESEARCH METHODS

This study is a quantitative case study using an analytical-numerical approach. The superstructure of an existing two-storey building is redesigned as a three-storey building. Structural responses are obtained from a three-dimensional finite element model, and structural members are designed and verified against the provisions of the current Indonesian standards. Standard design procedures are not described in detail but refer to the relevant standards. The overall research workflow is presented in Figure 1.

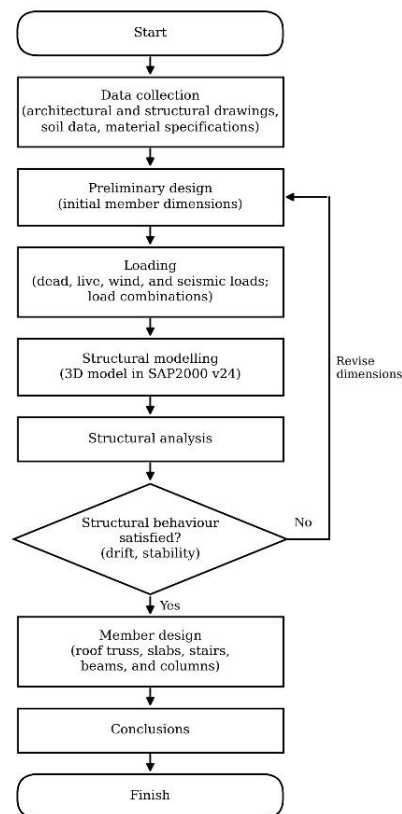


Figure 1. Flowchart of The Superstructure Redesign and Verification Procedure
Source: Authors' work (2026)

Research Location and Building Description

The case study building is the Religious Court Office Building of Penajam, located at Jl. Provinsi Km. 9, Islamic Center Complex, Nipah-Nipah Subdistrict, Penajam District, Penajam Paser Utara Regency, East Kalimantan Province, Indonesia. The site lies within the development area of Ibu Kota Nusantara (IKN). The main characteristics of the building are summarized in Table 1.

Table 1. Building Data

Parameter	Value
Function	Government office (religious court)
Number of storeys	3 (modified from 2)
Plan dimensions	37 m × 32 m
Storey height	4.5 m
Height to roof slab level	13.5 m <i>[verify against the stated total height of 16.5 m, which may include the roof truss]</i>
Structural system	Reinforced concrete special moment frame (SMF)
Roof structure	Steel truss (IWF rafters, cold-formed C purlins)

Data Collection

This study uses secondary data obtained from the planning consultant, CV. Panorama. The data comprise architectural drawings, structural drawings of the existing two-storey design, *[soil investigation report / SPT data used to determine the site class]*, and material specifications. The architectural drawings define the floor layout, room functions, and grid dimensions. The soil data are used to classify the site according to SNI 1726:2019. No primary field testing is carried out.

Materials and Design Standards

The concrete compressive strength is $f_c' = 25$ MPa. Deformed reinforcing bars have a yield strength of $f_y = 420$ MPa, and plain bars have $f_y = 280$ MPa *[see note below on transverse reinforcement]*. Structural steel for the roof truss is *[BJ 37 / BJ 41, $f_y = \dots$ MPa]*, and the cold-formed purlins are *[grade, $f_y = \dots$ MPa]*.

The following standards are used:

- SNI 1726:2019 for seismic design;
- SNI 2847:2019 for structural concrete;
- SNI 1727:2020 for minimum design loads, including wind loads;
- SNI 1729:2020 for hot-rolled structural steel;
- SNI 7971:2013 for cold-formed steel members.

Analysis Tools

Structural modelling and analysis are performed using SAP2000 v24 *[specify edition, e.g., Ultimate]*. Member design calculations and verification are carried out in Microsoft Excel spreadsheets developed by the authors in accordance with the standards listed in Section 3.4. Column axial–flexural capacities are checked using interaction diagrams generated.

Research Procedure

Preliminary design. Initial member dimensions are determined from the minimum thickness and depth requirements of SNI 2847:2019 and from the existing structural drawings. **Gravity loads.** Dead loads include the self-weight of structural members, which is calculated automatically by the software, and superimposed dead loads from finishes, ceilings, mechanical and electrical services, and partitions. Live loads are assigned according to room function based on SNI 1727:2020, as listed in Table 2.

Table 2. Live Loads Based on SNI 1727:2020

Occupancy	Live load (kN/m ²)
Offices	2.40
Corridors above first floor	3.83
Courtrooms (assembly, movable seats)	4.79
Archive rooms (light storage)	6.00
Ordinary roof	0.96

Seismic loads. The mapped spectral accelerations S_s and S_1 are obtained from the 2017 Indonesian seismic hazard maps adopted in SNI 1726:2019. The building is classified as Risk Category II with an importance factor of $I_e = 1.0$, and the site class is determined from the soil data as SD. The site coefficients F_a and F_v are applied to obtain SDS and SD_1 , from which the Seismic Design Category is determined. The seismic force-resisting system parameters for SMF are $R = 8$, $\Omega_0 = 3$, and $C_d = 5.5$. The equivalent lateral force procedure is applied, as permitted for the resulting Seismic Design Category. The fundamental period is limited by the upper bound $C_u T_a$. The seismic forces are applied at the centre of mass of each floor with an accidental eccentricity of 5% of the building dimension perpendicular to the direction of loading.

Wind loads. Wind loads on the main wind force-resisting system and on the roof components are calculated according to SNI 1727:2020, using a basic wind speed of $[m/s]$ and exposure category $[B/C]$. **Load combinations.** The load combinations follow SNI 1727:2020 and SNI 1726:2019 and include:

- $1.4D$;
- $1.2D + 1.6L + 0.5L_r$;
- $1.2D + 1.0W + L + 0.5L_r$;
- $0.9D + 1.0W$;
- $(1.2 + 0.2SDS)D + \rho QE + L$;
- $(0.9 - 0.2SDS)D + \rho QE$.

The redundancy factor ρ is taken as 1.0. Seismic forces are applied in the two principal directions using the 100%–30% orthogonal combination. **Structural modelling.** The structure is modelled as a three-dimensional frame. Beams and columns are represented by frame elements and slabs by shell elements. Each floor is assigned a rigid diaphragm, and the column bases are modelled as fixed supports. Cracked section properties are applied according to SNI 2847:2019: $0.35I_g$ for beams, $0.70I_g$ for columns, and $0.25I_g$ for slabs. **Member design.** The steel roof members are designed first, and their support reactions are transferred to the concrete frame. Slabs are then designed, followed by beams for flexure, shear, and torsion, and finally columns for combined axial force and bending, shear, and slenderness effects. The detailing of beams, columns, and beam–column joints follows the SMF provisions of Chapter 18 of SNI 2847:2019.

Performance Criteria

The structural design is considered acceptable when all criteria in Table 3 are satisfied.

Table 3. Performance Criteria

Aspect	Criterion	Reference
Member strength	$\phi R_n \geq R_u$ (demand-to-capacity ratio ≤ 1.0)	SNI 2847:2019; SNI 1729:2020
Inter-storey drift	$\Delta = C_d \cdot \delta_{xe} / I_e \leq \Delta_a = 0.020 h_{sx}$	SNI 1726:2019
P- Δ stability	Stability coefficient $\theta \leq 0.10$, or effects included in analysis	SNI 1726:2019

Column slenderness	$k \cdot l_u / r \leq 22$ for sway frames; otherwise moment magnification is applied	SNI 2847:2019
Strong column-weak beam	$\Sigma M_{nc} \geq 1.2 \Sigma M_{nb}$	SNI 2847:2019, Chapter 18
Slab and beam deflection	Within the limits of SNI 2847:2019 Table 24.2.2	SNI 2847:2019
Roof member deflection	$\leq L/240$	-

RESEARCH RESULTS AND DISCUSSION

Research Result

Seismic Parameters and Lateral Force Distribution

The seismic design parameters of the building are listed in Table 4. The values of $SDS = 0.127 \text{ g}$ and $SD1 = 0.129 \text{ g}$ place the building in Seismic Design Category B. The fundamental period used in the equivalent lateral force procedure is $T_a = 0.485 \text{ s}$, with an upper limit of $C_u T_a = 0.824 \text{ s}$. The seismic response coefficient is $C_s = 0.016$, and the total effective seismic weight is $W = 27,092.9 \text{ kN}$, resulting in a design base shear of $V = 431.3 \text{ kN}$.

Table 4. Seismic Design Parameters

Parameter	Value
Risk category / importance factor, I_e	II / 1.0
Site class	SD
PGA	0.061 g
S_s / S_1	0.1194 g / 0.0804 g
F_a / F_v	1.6 / 2.4
SMS / SM1	0.191 g / 0.193 g
SDS / SD1	0.127 g / 0.129 g
Seismic Design Category	B
$R / \Omega_0 / C_d$	8 / 3 / 5.5
$T_a / C_u T_a$	0.485 s / 0.824 s
C_s	0.016
W	27,092.9 kN
V	431.3 kN

The vertical distribution of the base shear is presented in Table 5. Because $T < 0.5 \text{ s}$, the distribution exponent is $k = 1$. The third floor level receives the largest lateral force (48.2% of V) because it has the second-largest weight at a relatively high elevation.

Table 5. Vertical Distribution of Seismic Forces

Level	Elevation (m)	W_x (kN)	$W_x \cdot h_x$ (kN·m)	C_{vx}	F_x (kN)
Roof	13.5	3,976.9	53,688.2	0.258	111.4
3	9.0	11,128.4	100,155.6	0.482	207.9
2	4.5	11,987.5	53,943.8	0.260	112.0
Total		27,092.9	207,787.6	1.000	431.3

The load combinations applied in the analysis are $1.4D$; $1.2D + 1.6L + 0.5L_r$; $1.2D + 1.0W + L + 0.5L_r$; $0.9D + 1.0W$; $1.225D + QE + L$; and $0.875D + QE$. The coefficients 1.225 and 0.875 correspond to $(1.2 + 0.2SDS)$ and $(0.9 - 0.2SDS)$, with $\rho = 1.0$.

Steel Roof Structure

The design results of the roof members are summarized in Table 6. The lipped channel purlin C 125×50×20×2.3 has a combined bending ratio of 0.21 and a deflection of 3.4 mm, below

the allowable 22.2 mm. The IWF 200×100×5.5×8 rafter has compact flange and web sections, a combined force ratio of 0.27, and a deflection of 3.6 mm, below the allowable 19.2 mm.

Table 6. Design Results of Roof Members

Member	Check	Demand	Capacity / limit	Ratio	Status
Purlin C 125×50×20×2.3	Bending (M_{ux} / M_{uy})	2.13 / 1.31 kN·m	-	0.21	OK
	Deflection	3.4 mm	22.2 mm	0.15	OK
Rafter IWF 200×100×5.5×8	Flange slenderness λ	6.25	$\lambda_p = 10.97$	-	Compact
	Web slenderness λ	29.45	$\lambda_p = 108.44$	-	Compact
	Flexure	37.63 kN·m	$[\phi]M_n = 47.12$ kN·m	0.80	OK
	Shear	[27.30 / 60.31] kN	$\phi V_n = 139.85$ kN	[]	OK
	Combined axial- flexure	-	-	0.27	OK
	Deflection	3.6 mm	19.2 mm	0.19	OK

Slabs and Stairs

The factored moments in the roof slabs obtained from the moment coefficient method are presented in Table 7. Slab type A (4.0 m × 4.0 m) produces the largest moments, with a support moment of 5.22 kN·m/m and a midspan moment of 2.11 kN·m/m in each direction. These values govern the reinforcement design.

Table 7. Factored Moments in Roof Slabs ($q_u = 6.28$ kN/m²)

Type	Lx (m)	Ly (m)	M _{tx} (kN·m/m)	M _{lx} (kN·m/m)	M _{ty} (kN·m/m)	M _{ly} (kN·m/m)
A	4.00	4.00	-5.22	2.11	-5.22	2.11
B	3.00	4.00	-3.90	1.75	-3.22	1.07
C	3.00	3.00	-2.94	1.19	-2.94	1.19
D	2.00	4.00	-2.08	1.03	-1.43	0.30
E	2.00	3.00	-1.91	0.90	-1.43	0.43
F	2.00	2.00	-1.31	0.53	-1.31	0.53
M	1.50	3.00	-1.17	0.58	-0.81	0.17
N	1.50	1.50	-0.73	0.30	-0.73	0.30
O	1.50	2.70	-1.16	0.57	-0.81	0.18

The slab and stair reinforcement is summarized in Table 8. The provided reinforcement Ø10-100 at supports gives $\phi M_n = 17.78$ kN·m/m, and Ø10-200 at midspan gives $\phi M_n = 9.15$ kN·m/m. Both exceed the governing moments of slab type A.

Table 8. Slab and Stair Reinforcement

Element	Thickness (mm)	q_u (kN/m ²)	Support reinforcement	Midspan reinforcement	Max $M_u / \phi M_n$ (kN·m/m)
Roof slab	120	6.28	Ø10-100 (X & Y)	Ø10-200 (X & Y)	5.22 / 17.78
Floor slab	120	8.80	Ø10-100 (X & Y)	Ø10-200 (X & Y)	17.78
Stair slab (slope 34°)	150	-	Ø13-100	Ø13-100	-
Landing slab	120	-	Ø13-100	Ø13-100	-

Beams

The flexural design results of the beams are listed in Table 9. All beam sections are tension-controlled ($\epsilon_t > 0.005$), and all demand-to-capacity ratios are below 1.0. The highest ratio occurs at the midspan of B2 (0.98), followed by the support of B1 (0.93).

Table 9. Flexural Reinforcement of Beams

Beam (b×h, mm)	Location	Mu (kN·m)	Tension bars	Compression bars	As (mm ²)	φMn (kN·m)	Mu/φMn
B1 (350×700)	Support	407.07	7D19	4D19	1,984.7	436.97	0.93
	Midspan	322.98	6D19	3D19	1,701.2	379.70	0.85
B2 (300×600)	Support	264.52	6D19	3D19	1,701.2	310.24	0.85
	Midspan	257.51	5D19	3D19	1,417.6	263.54	0.98
B3 (250×500)	Support	216.90	6D19	3D16	1,701.2	238.73	0.91
	Midspan	145.71	4D19	[2D19]	1,134.1	168.76	0.86
B4 (200×400)	Support	81.18	4D16	2D16	804.2	91.89	0.88
	Midspan	65.81	3D16	2D16	603.2	71.20	0.92
B5 (200×300)	Support	17.94	2D13	2D13	265.5	23.32	0.77
	Midspan	10.34	2D13	2D13	265.5	23.32	0.44

The shear reinforcement is summarized in Table 10. For all beams, the maximum spacing governs over the required spacing, so the provided stirrup spacing is controlled by the spacing limits.

Table 10. Shear Reinforcement of Beams

Beam	Location	Vu (kN)	Av (mm ²)	s required (mm)	s max (mm)	Provided
B1	Support	284.51	226.29	286.19	162.1	Ø12-100
	Midspan	281.67	226.29	291.32	162.1	Ø12-150
B2	Support	223.09	226.29	291.94	137.1	Ø12-125
	Midspan	207.07	226.29	331.60	137.1	Ø12-125
B3	Support	180.78	226.29	266.36	112.1	Ø12-100
	Midspan	177.64	226.29	273.53	112.1	Ø12-100
B4	Support	73.76	157.14	489.31	88.0	Ø10-75
	Midspan	72.03	157.14	514.23	88.0	Ø10-75
B5	Support	16.08	100.57	697.17	63.9	Ø8-50
	Midspan	16.08	100.57	697.17	63.9	Ø8-50

The torsion design results are listed in Table 11. For all beams, the required longitudinal torsion reinforcement A_t is smaller than the minimum $A_{t,min}$, so the minimum requirement governs. Beams B1 and B2 require 2D13, and beams B3–B5 require 2D10.

Table 11. Torsion Reinforcement of Beams

Beam	Tu [kN·m]	At/s (mm ² /mm)	A _t (mm ²)	A _{t,min} (mm ²)	Provided
B1	[48.99]	0.0003	0.85	248.74	2D13
B2	[31.30]	0.0003	0.66	85.37	2D13
B3	[31.67]	0.0005	0.85	28.00	2D10
B4	[15.20]	0.0004	0.55	55.78	2D10
B5	[3.21]	0.0001	0.13	125.68	2D10

Columns

The column design results are presented in Table 12. Column K1 (500×500 mm) carries the largest axial force, $P_u = 2,248.58$ kN, corresponding to an axial load ratio of $P_u/(A_g \cdot f'_c) = 0.36$. Column K1 is reinforced with 12D22 ($\rho = 1.82\%$), and column K2 (300×300 mm) with 8D13 ($\rho = 1.18\%$). Both ratios lie within the 1–6% range. The slenderness ratios with $k = 1.0$ are 25.3 for K1 and 44.4 for K2. Both exceed the sway-frame limit of 22, so second-order effects are included through the moment magnifier δ_s .

Table 12. Column Design Results

Column (mm)	Pu (kN)	M1 (kN·m)	M2 (kN·m)	k·lu/r	δs	Mc (kN·m)	Reinforcement	ρ (%)	$\phi P_{n,max}$ (kN)
K1 (500×500)	2,248.58	140.62	205.79	25.3	-	-	12D22	1.82	3,708.3
K2 (300×300)	242.37	6.82	10.33	44.4	-	-	8D13	1.18	1,214.7

The shear reinforcement of the columns is listed in Table 13. For column K2, the concrete shear capacity exceeds the shear demand ($V_c > V_u/\phi$), so the minimum transverse reinforcement requirement governs.

Table 13. Shear Reinforcement of Columns

Column	Vu (kN)	Vc (kN)	s required (mm)	s max (mm)	Provided
K1	180.28	259.29	290.14	-	Ø12-100
K2	2.15	62.93	— ($V_c > V_u/\phi$)	-	Ø10-100

Discussion

The results show that the building falls into Seismic Design Category B, with a design base shear of only 431.3 kN, or 1.6% of the effective seismic weight. This low value follows directly from the low mapped spectral accelerations at the site ($S_s = 0.1194$ g and $S_1 = 0.0804$ g) and from the response modification coefficient $R = 8$ assigned to special moment frames. Because the seismic response coefficient is obtained by dividing SDS by R/I_e , the lateral design force of an SMF building in a low-seismicity region becomes very small compared with its gravity loads. This finding is consistent with regional hazard studies. The probabilistic seismic hazard analysis of the new capital region classified its disaster hazard index as low, and Khalqillah et al. (2025) similarly found lower overall seismic hazard in Kalimantan than in Sumatra.

However, the same probabilistic analysis reported maximum bedrock spectral accelerations of 0.4–0.5 g at 0.2 s for a 2,500-year return period in the new capital region (Mountainhia et al., 2022). This is considerably higher than the S_s value of 0.1194 g adopted in this study from the SNI 1726:2019 hazard map. Khalqillah et al., (2025) also noted localized high intensities in East Kalimantan caused by the Meratus and Mangkalihat faults. The difference shows that the code-based parameters used here represent the current regulatory minimum, and that seismic demand in the IKN region may increase if future revisions of the national hazard map incorporate these site-specific findings. Code revisions have already been shown to change seismic demand significantly: Sarya et al., (2021) reported that the transition from SNI 1726:2012 to SNI 1726:2019 increased base shear by 47% and 69% in the two directions of a building in Malang.

The low seismic demand also explains why most members are governed by minimum and detailing requirements rather than by strength demand. The roof and floor slabs have demand-to-capacity ratios of only 0.29 at supports and 0.23 at midspan, so their reinforcement is controlled by minimum reinforcement and maximum spacing provisions. For all beams, the required stirrup spacing from shear demand (266–697 mm) is larger than the maximum spacing permitted by SNI 2847:2019 (64–162 mm), and all torsional longitudinal reinforcement is controlled by $A_{l,min}$. Only flexural reinforcement is governed by strength, with demand-to-capacity ratios between 0.44 and 0.98. When lateral forces are small, internal forces are dominated by gravity loads, and the provisions intended to ensure ductility and crack control become the controlling requirements. This finding agrees with Ramadhani et al., (2026), who designed a building in East Kalimantan with both intermediate and special moment frames and found identical member dimensions, with differences only in the closer transverse

reinforcement spacing of the SMF. Together, the two studies indicate that in low-seismicity regions of East Kalimantan, the choice of moment frame system mainly affects detailing rather than member sizes. The revision of beam B4 at midspan from 2D16 to 3D16, which reduced its demand-to-capacity ratio from 1.34 to 0.92, further shows that secondary beams with small cross-sections still require individual verification, because their demand relative to capacity can exceed that of the main beams.

With respect to the columns, K1 carries an axial force of 2,248.58 kN, corresponding to 61% of its maximum axial capacity of 3,708.3 kN and an axial load ratio of 0.36. The slenderness ratios of both columns (25.3 for K1 and 44.4 for K2) exceed the sway-frame limit of 22. This results from the relatively large storey height of 4.5 m, which is typical of public buildings with courtrooms and service halls, so second-order effects cannot be neglected. These results are relevant for comparison with previous vertical extension studies. Kurniawan et al., (2024) found that the existing 250 × 400 mm columns of a two-storey building could not resist the loads after extension to three storeys, because the load points fell outside the interaction diagram. Bahrami et al., (2022) likewise concluded that several load-bearing elements needed strengthening before a vertical extension could proceed. In contrast, Subagio et al., (2021) found that a lecture building remained safe after the addition of three storeys. The present study differs from these works because the columns were redesigned for the three-storey configuration rather than evaluated as existing members. The results indicate that K1 has sufficient axial reserve for the extended building. However, if the third storey is built on columns constructed for the original two-storey design, their capacity must be verified against the demands obtained here, as demonstrated by Kurniawan et al., (2024).

The small lateral forces and the relatively large member dimensions required by gravity loads indicate that the lateral stiffness of the frame is governed by gravity design. This contrasts with buildings in high-seismicity regions, where lateral demand typically controls member sizes. In vertical extension studies, Subagio et al., (2021) similarly reported that the inter-storey drift of an extended building remained well within the allowable limit. Particular attention was nevertheless given to capacity design requirements, because field observations after the 2023 Kahramanmaraş earthquakes identified strong beam weak column mechanisms and insufficient transverse reinforcement as major design-stage causes of damage (Altunışık et al., 2023). These findings answer the research question of this study: the superstructure can be safely redesigned for three storeys using the member dimensions and reinforcement presented in the results. From a practical perspective, this study documents that for a low-rise government building in the IKN region, the design is controlled mainly by gravity loads, minimum reinforcement, and SMF detailing provisions rather than by seismic strength demand. The selection of the seismic force-resisting system in such regions could therefore be evaluated in terms of constructability and cost, since the use of SMF primarily increases transverse reinforcement and detailing effort. The study also highlights the sensitivity of the design to the adopted hazard parameters, given the gap between the current code map and site-specific hazard results for the region.

This study is limited to the superstructure, and the foundation capacity under the increased loads was not evaluated. The structure was modelled as monolithic, whereas Shala & Bleiziffer (2024) showed that connections between existing structures and added storeys may not behave rigidly; the actual joint behaviour therefore requires further study if the extension is built on the existing structure. The analysis used the linear equivalent lateral force procedure, and nonlinear performance was not assessed. Future studies are recommended to evaluate foundation capacity, compare SMF and IMF alternatives in terms of material quantity and cost, and perform pushover analysis to assess the seismic performance of the extended building.

CONCLUSION

The superstructure of the Religious Court Office Building of Penajam was successfully redesigned from two to three storeys in accordance with SNI 1726:2019, SNI 2847:2019, SNI 1727:2020, and SNI 1729:2020. The building is classified as Seismic Design Category B, with a design base shear of 431.3 kN, or 1.6% of the effective seismic weight. The roof structure uses C 125×50×20×2.3 purlins and IWF 200×100×5.5×8 rafters. The 120 mm roof and floor slabs are reinforced with Ø10-100 at supports and Ø10-200 at midspan, and the 150 mm stair slab with Ø13-100. The main beam B1 (350×700 mm) requires 7D19 tension reinforcement at the supports and 6D19 at midspan, with Ø12-100 and Ø12-150 stirrups. Column K1 (500×500 mm) requires 12D22 with Ø12-100 ties, and column K2 (300×300 mm) requires 8D13 with Ø10-100 ties. All members satisfy the strength requirements, with a maximum flexural demand-to-capacity ratio of 0.98. The main finding of this study is that, for a low-rise government building in the low-seismicity IKN region, the design is controlled mainly by gravity loads, minimum reinforcement, and special moment frame detailing provisions rather than by seismic strength demand. This is shown by the low demand-to-capacity ratios of the slabs and by the governing role of maximum stirrup spacing and minimum torsional reinforcement in all beams. In such regions, the selection of the seismic force-resisting system therefore has a greater influence on detailing and constructability than on member dimensions.

This study has several limitations. The analysis covered only the superstructure and did not evaluate foundation capacity under the increased loads. The structure was modelled as a monolithic frame without considering the behaviour of connections between existing and new elements. The analysis used the linear equivalent lateral force procedure, and inter-storey drift, P-Δ stability, and strong column-weak beam requirements were not presented in detail. Further research is recommended to verify these seismic performance criteria, evaluate the foundation, compare special and intermediate moment frames in terms of material quantity and cost, and assess the nonlinear performance of the building through pushover analysis. Such studies would also benefit from considering site-specific seismic hazard parameters for the IKN region.

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