Structural Redesign For Material Efficiency (Case Study: Building A Of Kiara Beach Front Apartment)

Made Hendra Prayoga[1], Ida Bagus Prastha Bhisama[1], P. Adi Yasa[1]
Department of Civil Engineering Faculty of Engineering Universitas Udayana, Badung, Bali[1]
*Corresponding author e-mail address: hendra.prayoga@unud.ac.id

ABSTRACT

Structural design for multistory buildings must satisfy not only strength and safety requirements but also material efficiency to achieve cost-effective construction. This study aims to redesign the structural system of Building A of the Kiara Beach Front Apartment to obtain a more efficient design while maintaining structural performance and safety. The redesign was carried out using the Special Moment Resisting Frame (SMRF) system in accordance with the provisions of SNI 1726:2019, SNI 1727:2020, and SNI 2847:2019. A three-dimensional structural model was developed in SAP2000, followed by analyses of gravity and seismic loads, inter-story drift, structural irregularities, and the design of beams, columns, slabs, and foundations. The results indicate that the redesigned structure satisfies seismic performance requirements, with the fundamental period and inter-story drift remaining within the allowable limits specified by the applicable standards. Furthermore, the optimization of structural member dimensions and reinforcement improved material efficiency in the first-floor structural elements, resulting in an 11% reduction in concrete volume and a 23% reduction in reinforcement weight. Consequently, a more economical structural design was achieved without compromising structural safety requirements.

Keywords: Material efficiency, Reinforced concrete, Resisting Frame (SMRF), SAP2000, Special Moment, Structural redesign.

1. INTRODUCTION

The rapid growth of the construction industry, driven by increasing demand for modern infrastructure, has accelerated the development of high-rise buildings, including apartment buildings for residential and investment purposes. In Bali Province, the property sector continues to expand, supported by strong economic and tourism activities [1]. Consequently, structural design is expected not only to satisfy safety requirements but also to achieve efficient material utilization.

Structural efficiency is a key consideration in the design of reinforced concrete buildings because it directly affects material consumption, construction costs, and constructability. In Building A of Kiara Beach Front Apartment, the column dimensions and reinforcement details are identical from the basement to the top floor, although the axial forces and bending moments acting on the columns theoretically decrease with increasing building height due to the reduced accumulation of gravity loads [2]. In addition, the building includes a beam with a span of 9.55

m and a depth of 650 mm, which exceeds the preliminary span-to-depth ratio recommendation of L/18.5 [3], indicating the potential for further optimization of structural member dimensions.

The building is located in a region with high seismic hazard and is classified as Risk Category II and Seismic Design Category (SDC) D in accordance with SNI 1726:2019. Therefore, it is designed using a Special Moment Resisting Frame (SMRF) system to ensure adequate seismic performance [4]. Several studies on the redesign of reinforced concrete SMRF structures have demonstrated that optimizing member dimensions and reinforcement can produce more efficient structural designs while maintaining compliance with applicable design standards [5], [6]. Based on these considerations, this study redesigns the structural system of Building A of Kiara Beach Front Apartment using the same structural system and seismic design criteria as the original design. The optimized design is then compared with the existing design to evaluate the achievable structural efficiency without compromising structural safety and performance.

2. THEORY AND METHODS

2.1 Theory

2.1.1 Reinforced Concrete

Reinforced concrete is a composite material consisting of concrete and reinforcing steel acting together to resist applied loads. Concrete primarily resists compression, whereas reinforcing steel carries tensile forces [7]. The design of beams and columns follows the provisions of SNI 2847:2019, where flexural strength, shear strength, and reinforcement requirements are determined based on structural analysis.

2.1.2 Story Drift

Interstory drift (Δ) is defined as the relative lateral displacement between two consecutive floors of a structure [8]. According to Clause 7.8.6 of SNI 1726:2019, the interstory drift is determined as the difference in displacement between the centers of mass of two adjacent stories. If the centers of mass are not vertically aligned, the displacement is calculated based on their vertical projection.

$$\delta_x = \frac{Cd.\delta_{xe}}{Ie} \tag{3}$$

The allowable interstory drift limit for Risk Category II buildings is specified as (0.025hsx), where (hsx) is the story height, as stipulated in Table 20 of SNI 1726:2019.

2.1.3 Structural Irregularity

According to Clause 7.3.2 of SNI 1726:2019, buildings shall be classified as either regular or irregular based on their horizontal and vertical configurations. Horizontal irregularities are defined in Clause 7.3.2.1 and identified according to the criteria presented in Table 13 of SNI 1726:2019. Similarly, vertical irregularities are specified in Clause 7.3.2.2 and categorized according to Table 14 of the same standard. Buildings exhibiting one or more irregularity types must comply with the corresponding design requirements referenced in the applicable clauses.

2.1.4 Structural Loading

The structural loading was assigned in accordance with the provisions of SNI 1727:2020 Table 4.3-1 [9] and PPPURG 1987 [10]. The applied loads consisted of the structure's self-weight, superimposed dead loads, live loads, and seismic loads in accordance with SNI 1726:2019. The superimposed dead loads included floor finishing and mechanical, electrical, and plumbing (MEP) system loads. The live loads comprised floor occupancy loads and rain loads acting on the rooftop.

2.1.5 Seismic Loading Procedure

The seismic analysis procedure follows Clause 7.6 and Table 16 of SNI 1726:2019. The analysis begins with an evaluation of structural irregularities. If the building is classified as regular, the Equivalent Lateral Force (ELF) procedure may be adopted. Otherwise, if structural irregularities are identified, a dynamic analysis procedure is required to accurately evaluate the seismic response.

2.1.6 Foundation

According to [11] and [12], the foundation is a structural component that safely transfers superstructure loads to the supporting soil while maintaining the overall stability of the structure. A pile cap is a structural element that connects the column to the pile group and distributes the applied loads to the piles according to the pile arrangement [13]. The center-to-center spacing between piles is generally specified as 2.5–3 times the pile diameterr, while the distance from the pile center to the edge of the pile cap should be 1–1.5 times the pile diameter [14]. Pile foundations transfer structural loads to deeper, more competent soil layers, with the ultimate axial bearing capacity calculated using Equation (4).

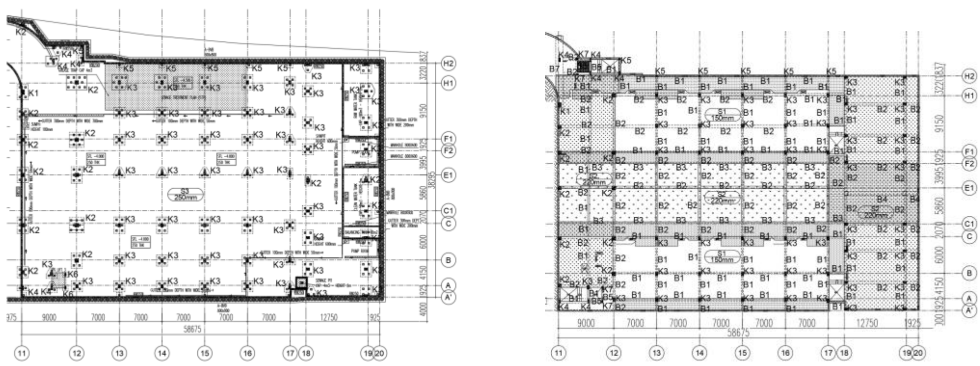
$$Q_u = Q_b + Q_s \tag{4}$$

2.2 Methods

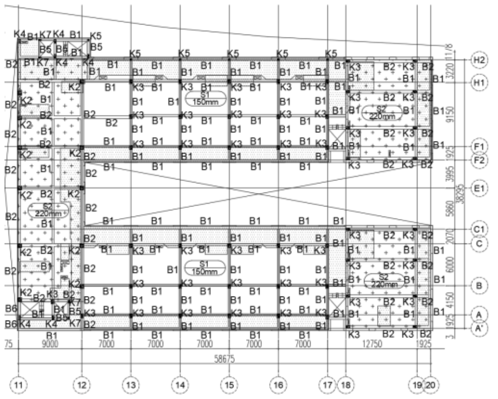
The structural redesign of Building A at Kiara Beach Front Apartment commenced with the preliminary sizing of structural members, followed by structural modeling and load analysis using SAP2000. The analysis results were then evaluated by examining structural irregularities, stress ratios, inter-story drift, and the fundamental structural period to verify compliance with the applicable structural design provisions.

2.2.1 Design Data

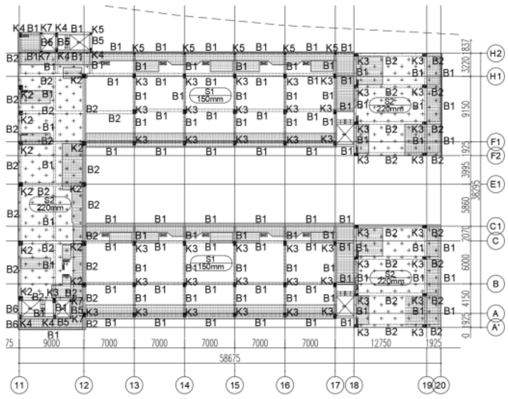
Building A of Kiara Beach Front Apartment is located at Jalan Pratama No. 80, Tanjung Benoa, Badung, Indonesia. The building has a footprint area of 3,700 m² and consists of four residential stories and one basement used for parking. The story height is 3.6 m, the basement depth is 4.0 m, and the total building height is 18.4 m. The structural system is designed using reinforced concrete with a specified concrete compressive strength (f'c) of 27.5 MPa and reinforcing steel with a yield strength (fy) of 420 MPa. A reinforced concrete roof slab is adopted as the roof structural system.



(a) (b)



(c)



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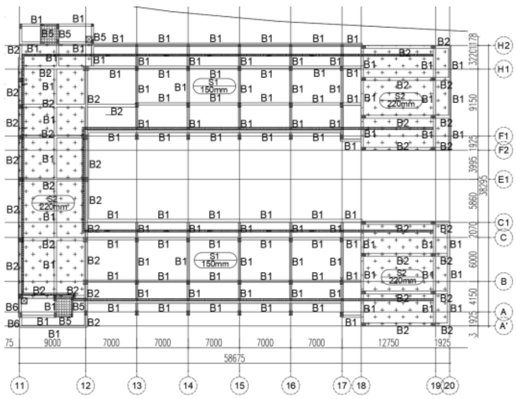


(e)

Figure 1. Floor plan layouts: (a) Basement, (b) First Floor, (c) Second and Third Floors, (d) Fourth Floor, and (e) Rooftop.

2.2.2 Initial Member Dimensions

The initial dimensions of the structural members were established based on the existing dimensions of Building A of the Kiara Beach Front Apartment.

Table 1. Existing Structural Dimensions

Element Structural	Dimensions Existing (mm)		Element Structural	Dimensions Existing (mm)	
	B	H		B	H
Slab S1	150		K6	200	600
Slab S2	220		K7	200	400
Slab S3	250		B1	300	500
Retaining Wall	250		B2	400	650
K1	600	800	B3	200	650
K2	600	600	B4	600	650
K3	400	600	B5	200	500
K4	300	600	B6	400	500
K5	300	300	B7	800	1200

2.2.3 Structural Modeling and Loading

The structural model was developed using SAP2000 v22 based on the architectural floor plans and material properties. Beams and columns were modeled as frame elements, while slabs and retaining walls were represented using shell elements. The foundation system was idealized as pinned supports. The loading system considered in the analysis consisted of dead load, superimposed dead load, live load, seismic load, and lateral earth and water pressures. The dead load was automatically calculated by SAP2000 using the "Dead Load" load pattern based on the dimensions and material properties of the structural members.

The superimposed dead load was assigned manually using the "Super Dead" load pattern and included floor finishing materials as well as mechanical, electrical, and plumbing (MEP) components. Superimposed dead loads applied in the model are summarized in Table 2.

Table 2. Superimposed Dead Loads

Load Component	Value	Unit
Floor finishing	145	kg/m²
Basement floor finishing	63	kg/m²
Roof finishing	38	kg/m²
Elevator load (10-person) [15]	6.938	kg/m²
Basement staircase	252,6	kg/m²
Staircase (Floors 1–4)	265,8	kg/m²
150-mm masonry wall (Basement)	288	kg/m
150-mm masonry wall (Floors 1–4)	259,2	kg/m
Landscape soil (300 mm thick)	5,4	kN/m²

The live loads were manually defined under the **"Live"** load pattern as follows:

Table 3. Live Loads

Loading Area	Load	Loading Area	Load
Parking area	1,92 kN/m²	Rain load	39,2 kg/m²
Corridor	4,79 kN/m²	Roof live load	100 kg/m²
Public area	4,79 kN/m²	Staircase	4,79 kN/m²
Residential unit	1,92 kN/m²	Elevator load (10-person) [15]	700 kg

The seismic load was assigned using the "Quake" load pattern through the Auto Lateral Load feature. The seismic parameters were defined based on the project location in Tanjung Benoa, Badung, with a mapped short-period spectral acceleration of (S_S = 0.9601) and a mapped 1-second spectral acceleration of (S_1 = 0.4027). The structure was designed using a response modification factor of (R = 8), Risk Category II, and a seismic importance factor of (I_e = 1.0). Site Class SC was adopted, with site coefficients of (F_a = 1.2) and (F_v = 1.5). Based on a design spectral acceleration of (S_{DS} = 0.7681), the structure was classified as Seismic Design Category D (SDC D) and designed as a Special Moment Resisting Frame (SMRF) system.

The lateral earth pressure was calculated using a soil unit weight (γ_s) of 20.4 kN/m³, while the lateral hydrostatic pressure was determined using a water unit weight (γ_w) of 9.81 kN/m³. The resulting lateral earth and water pressures are summarized in Table 4.

Table 4. Lateral Earth and Water Pressures

Depth (m)	Active Earth Pressure (kN/m²)	Hydrostatic Water Pressure (kN/m²)
1	5.528	9.81
2	11.056	19.62
3	16.585	29.43
4	22.113	39.24

2.2.4 Foundation Design

The foundation system was designed based on the soil bearing capacity and the maximum column load. The design process included the determination of the allowable pile capacity, the required number of piles, and the pile group efficiency. Subsequently, the dimensions and reinforcement of the pile cap were designed considering the column load as well as the tensile, flexural, and shear forces acting on the pile cap.

3. RESULTS AND DISCUSSION

3.1 Structural Irregularity Assessment

Based on the evaluation results presented in Tables 5 and 6, the building exhibits Type 2 and Type 4 horizontal structural irregularities as well as a Type 2 vertical structural irregularity.

Table 5. Re-entrant Corner Irregularity

Lx (m)	Px (m)	Ly (m)	Py (m)	Px/Lx	Py/Ly	Irregularity (>15%)
58.55	49.70	41.03	24.30	85%	59%	Yes

Table 6. Mass (Weight) Irregularity

Story	Weight (kN)	150% of Lower Story (n−1)	150% of Upper Story (n+1)	>150% of Lower Story?	>150% of Upper Story?
Roof	9,494.45	23,580.8	–	No	–
4	15,720.5	26,992.4	14,241.7	No	Yes
3	17,994.9	27,569.4	23,580.8	No	No
2	18,379.6	69,281.5	26,992.4	No	No
1	46,187.7	–	27,569.4	–	Yes

Consequently, the structural design was carried out in accordance with the provisions of SNI 1726:2019, including the consideration of increased design forces for elements affected by structural irregularities, the inclusion of accidental torsional eccentricity, the design of discontinuous frame support elements using the overstrength factor ($\Omega_0 = 3$), three-dimensional structural modeling, and the application of response spectrum analysis as the required seismic analysis procedure

3.2 Story Drift Control

Table 7. Interstory Drift in the X-Direction

Story	*h*sx (mm)	δei (mm)	δi (mm)	Δi (mm)	Δa (mm)	Δi ≤ Δa
Roof	3600	18.4299	101.4	21.21	90.0	ok
4	3600	14.5741	80.2	24.40	90.0	ok
3	3600	10.1385	55.8	28.2	90.0	ok
2	3600	5.0022	27.5	27.1	90.0	ok
1	4000	0.0801	0.4	0.4	100.0	ok

Table 8. Interstory Drift in the Y-Direction

Story	*h*sx (mm)	δei (mm)	δi (mm)	Δi (mm)	Δa (mm)	Δi ≤ Δa
Roof	3600	24.6115	135.4	27.2	90.0	ok
4	3600	19.6625	108.1	36.0	90.0	ok
3	3600	13.1205	72.2	38.9	90.0	ok
2	3600	6.0548	33.3	32.7	90.0	ok
1	4000	0.1130	0.6	0.6	100.0	ok

The analysis results indicate that the maximum interstory drift occurred at the third floor in the Y-direction, with a displacement of 38.9 mm. Overall, the interstory drifts in both principal directions satisfy the allowable limits specified in SNI 1726:2019, indicating that the structure complies with the code requirements and does not exhibit a soft-story irregularity.

3.3 Modal Mass Participation

The cumulative modal mass participation exceeded the minimum requirement of 90% specified in SNI 1726:2019, reaching 93% in the X-direction and 92% in the Y-direction. Therefore, the modal analysis satisfies the code requirements.

3.4 Fundamental Period and Base Shear Scaling

The fundamental periods obtained from the SAP2000 analysis were 0.686 s in the X-direction and 0.671 s in the Y-direction. Both values fall within the allowable range specified by SNI 1726:2019, i.e., between the approximate period (Ta = 0.641 s) and the upper limit (Tmax = 0.897 s).

The initial scale factor was calculated as 1 × g / R = 1.226 m/s². Based on the SAP2000 response spectrum analysis, the base shear correction factors were determined as 1.7956 for the X-direction (6,526,720.93 / 3,634,920.56) and 1.8669 for the Y-direction (6,677,920.59 / 3,576,980.03). These correction factors were subsequently applied to the initial scale factor to ensure compliance with the seismic design requirements. Consequently, the final modal response spectrum analysis produced a dynamic base shear (Vt) equal to 100% of the equivalent static base shear (V).

3.5 Cross-Section Capacity Ratio Results

Figure 2 shows the cross-section capacity ratio evaluation of the redesigned structure. The analysis results indicate that none of the structural members experience overstress, as all demand-to-capacity ratios remain within the allowable limits. This demonstrates that the redesigned structural dimensions satisfy the strength requirements while maintaining adequate structural safety.

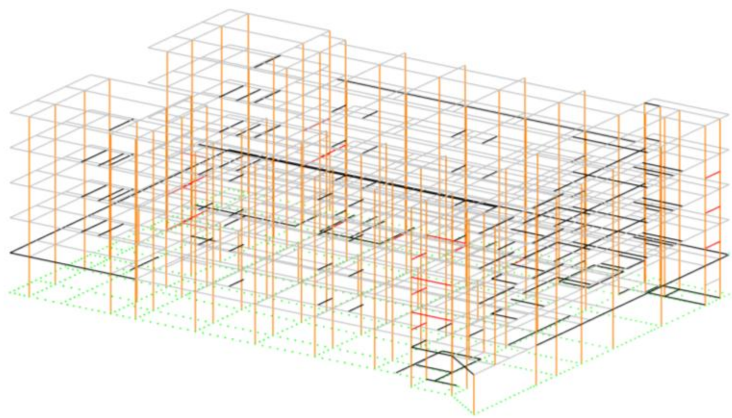


Figure 2. Cross-Section Capacity Ratio Evaluation of the Redesigned Structure.

3.6 Beam Design

Beam B2 (400 × 550 mm) on the second floor of Frame 476 was selected as the representative primary beam for structural evaluation. The geometric requirements for the Special Moment Resisting Frame (SMRF) were verified, indicating that the effective depth, d = 487.5 mm, satisfied the requirement of $Ln \geq 4d$, while the beam width also met the minimum criteria of $0.3h$ and 250 mm. Flexural reinforcement design resulted in a single-layer reinforcement arrangement, with the maximum required reinforcement area occurring at the top support (833.934 mm²). The required shear reinforcement area was determined to be 265.465 mm², whereas torsional analysis required a maximum reinforcement area of 1,809.483 mm². The complete reinforcement detailing for Beam B2 is presented in Figure 3.

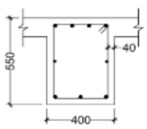
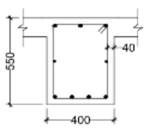
BEAM	B2 (400X550)	
	SUPPORT	MIDSPAN
		
TOP REBAR	4 D19	3 D19
BOT REBAR	3 D19	4 D19
WEB REBAR	2 D13	2 D13
STIRRUPS	2 D13-100	2 D13-140

Figure 3. Reinforcement Details of Beam B2

3.7 Column Design

Column K2 (600 × 600 mm) was analyzed using spColumn under various load combinations. The interaction diagram indicates that all loading conditions fall within the safe region, demonstrating adequate strength capacity. The required longitudinal reinforcement area is 12,868 mm², corresponding to a reinforcement ratio of 3.56%. The required transverse reinforcement ratio is Av/s = 3.685 mm²/mm within the plastic hinge region and Av/s = 0.662 mm²/mm outside the plastic hinge region. Verification of the Strong Column–Weak Beam (SCWB) criterion yielded an SCWB ratio of 0.191, satisfying the design requirement of $\Sigma Mnc \geq 1.2\Sigma Mnb$. The complete reinforcement detailing of Column K2 is presented in Figure 4.

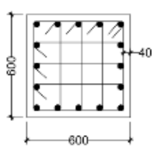
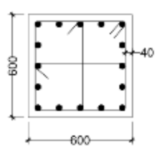
COLUMN	K2 (600X600)	
	SUPPORT	MIDSPAN
		
DIMENSION	600 X 600	600 X 600
REBAR	16 D32	16 D32
STIRRUPS	2D10-100 + 6 Ties	2D10-150 + 2 Ties

Figure 4. Reinforcement detailing of Column K2

3.8 Beam–Column Joint Design

The beam–column joint was designed in accordance with SNI 2847:2019 by evaluating the joint shear capacity under the beam-hinging mechanism. The joint, with an effective dimension of 600 × 600 mm, was classified as an unconfined joint (Configuration 3). The analysis indicated that the design joint shear capacity of 1,604.68 kN exceeded the applied joint shear demand, resulting in safety factors of 2.119 and 1.914 in the X and Y directions, respectively. These results confirm that the joint satisfies the required shear strength provisions. In addition, the required tensile reinforcement development length of 300 mm complies with the anchorage requirements specified in SNI 2847:2019.

3.9 Floor Slab Design

Based on the SAP2000 analysis results, the S2 floor slab (200 mm thick) at the first floor was designed with a 25 mm concrete cover and Ø10 mm main reinforcement. The effective depths were calculated as (d_x = 170) mm and (d_y = 160) mm. Structural evaluation indicated that the slab did not experience overstress, confirming that the design satisfies the required strength criteria. The slab aspect ratio was determined as (L_y/L_x = 0.88 < 2), classifying the slab as a two-way slab. In addition, the reinforcement ratio satisfied the requirements of SNI 2847:2019, with (0.0033 < \rho < 0.021). The reinforcement design results indicate that the provided reinforcement is adequate and can be adopted as detailed in Figure 5.

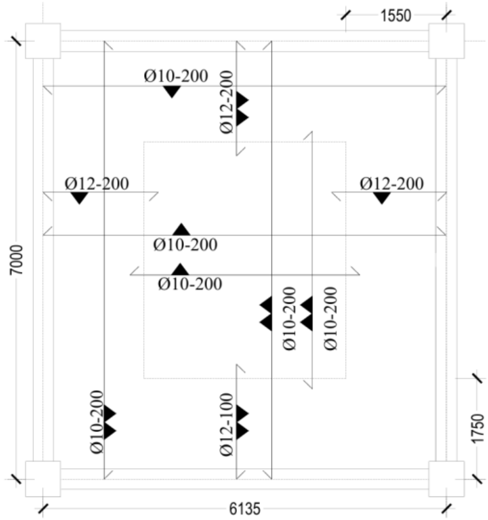


Figure 5. Floor Slab Reinforcement Details

3.10 Basement Wall Design

Based on the SAP2000 analysis results, the RW1 retaining wall, with a thickness of 250 mm, was designed using a 25 mm concrete cover and Ø13 mm main reinforcement. The effective depths were calculated as 219 mm in the x-direction and 207 mm in the y-direction. The structural evaluation indicated that the retaining wall satisfied the strength requirements without experiencing overstress. The aspect ratio analysis yielded $L_y/L_x = 1.75 < 2$, classifying the wall panel as a two-way slab. The provided reinforcement ratio satisfied the requirements of SNI 2847:2019, with $0.0033 < \rho < 0.021$. The final reinforcement design consisted of Ø13-100 mm for both vertical support and vertical span reinforcement, Ø10-200 mm for both horizontal support and horizontal span reinforcement, and Ø10-200 mm distribution and shrinkage reinforcement.

3.11 Foundation Design

The foundation design was performed based on the results of the Standard Penetration Test (SPT). A 22 m deep driven pile foundation was selected for the building. The analysis indicated that nine piles with cross-sectional dimensions of 300 × 300 mm and a center-to-center spacing of 900 mm provide an allowable axial bearing capacity of 1,497 kN, which exceeds the maximum axial load acting on the foundation. Subsequently, the pile cap was designed by evaluating the governing shear forces and bending moments. The final pile cap dimensions were 2.7 m × 2.7 m with a thickness of 750 mm and a 75 mm concrete cover. The required reinforcement consists of D19 bars spaced at 200 mm.

3.12 Structural Comparison

The comparison between the existing structure and the redesigned structure was conducted based on the concrete volume and reinforcement weight of the first floor, as this level is subjected to the highest structural demand due to the cumulative gravity loads from all upper floors.

Table 9. Comparison of Concrete Volume Between the Existing and Redesigned Structures

Structural Member Code		Concrete Volume (m^3)		V_{design} / $V_{existing}$
Existing Design	**Redesigned Structure**	**Existing Design**	**Redesigned Structure**	
K2	K2	14.26	14.26	1.00
K3	K3	41.47	20.74	1.08
	K3A		17.28	
	K3D		6.91	
K4	K4	3.89	3.89	1.00
K5	K5	1.94	1.62	1.13
	K5A		0.58	
K7	K7	1.15	1.15	1.00
B1	B1	59.20	47.61	0.80
B2	B2	203.32	148.92	0.80
	B2A		11.84	
	B2C		1.33	
B3	B3	10.58	6.57	0.62
B4	B4	4.47	2.26	0.56
	B4A		0.22	
B5	B5	0.99	0.77	0.78
B7	B7	10.26	4.83	0.47
Slab S2		270.41	245.83	0.91
Slab S1		160.10	160.10	1.00
Total		**782.05**	**696.71**	**0.89**

The comparison results indicate that most structural members experienced a reduction in concrete volume after the redesign. The greatest reduction was observed in beam B7, with a volume ratio of 0.47, whereas ratios greater than 1.00 indicate an increase in concrete volume, with the largest increase occurring in column K5, which had a ratio of 1.13. Overall, the total concrete volume decreased from 782.05 m³ in the existing design to 696.71 m³ in the redesigned structure, corresponding to an 11% reduction in concrete consumption.

Table 10. Comparison of Reinforcement Weight Between the Existing and Redesigned Structures.

Structural Member Code		Reinforcement Weight (ton)		V_{design} / $V_{existing}$
Existing Design	**Redesigned Structure**	**Existing Design**	**Redesigned Structure**	
K2	K2	5.90	6.25	1.06
K3	K3	19.10	9.53	0.98
	K3A		7.01	
	K3D		2.18	
K4	K4	1.70	1.44	0.85
K5	K5	0.83	0.75	1.17
	K5A		0.22	
K7	K7	0.34	0.65	1.88
B1	B1	12.36	7.91	0.64
B2	B2	45.80	24.26	0.58
	B2A		2.10	
	B2C		0.22	
B3	B3	2.27	0.89	0.39
B4	B4	1.05	0.24	0.26
	B4A		0.04	
B5	B5	0.13	0.30	2.22
B7	B7	1.64	0.64	0.39
Slab S2		8.50	8.50	1.00
Slab S1		7.36	7.36	1.00
Top Additional Slab Reinforcement		3.59	3.51	0.98
Bottom Additional Slab Reinforcement		7.38	6.61	0.90
Total		**117.97**	**90.59**	**0.77**

Based on the comparison results, most structural members exhibited a reduction in reinforcement requirements after the redesign. The highest reinforcement efficiency was achieved in beam B4, with a reinforcement ratio of 0.26, whereas a ratio greater than 1.0 indicates an increase in reinforcement demand. The largest increase occurred in beam B5, with a ratio of 2.22. Overall, the total reinforcement weight was reduced from 117.97 t to 90.59 t, corresponding to a 23% reduction compared with the existing structural design.

4. CONCLUSIONS

Building A of the Kiara Beach Front Apartment is a six-story reinforced concrete building that was redesigned using a Special Moment Resisting Frame (SMRF) system. The main findings of this study are summarized as follows:

1. The maximum interstory drift was 28.2 mm in the X-direction and 38.9 mm in the Y-direction, both of which are below the allowable limit of 90.0 mm specified by SNI 1726:2019.
2. The fundamental period of the redesigned structure was 0.686 s, which is lower than the maximum allowable period of 0.897 s specified in SNI 1726:2019.
3. The building exhibits Type 2 and Type 4 horizontal irregularities and a Type 2 vertical irregularity.
4. Beams and columns located within the discontinuous column region were designed using the overstrength factor (Ω_0 = 3) in accordance with the seismic design requirements.

5. The redesigned beams, columns, and beam–column joints satisfy the requirements for a Special Moment Resisting Frame (SMRF), including the Strong Column–Weak Beam (SCWB) criterion.
6. The foundation system consists of 300 × 300 mm precast driven piles and a 2700 × 2700 × 750 mm pile cap, which satisfies the shear strength requirements.
7. Overall, the redesigned first-floor structural members achieved an 11% reduction in concrete volume, from 782.05 m³ to 696.71 m³, and a 23% reduction in total reinforcement weight, from 117.97 t to 90.59 t, demonstrating a more efficient structural design while maintaining compliance with the applicable design standards.

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