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Seismic-Resistant Superstructure Redesign of an Apartment Building Based on SNI 1726:2019 in Semarang City

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Abstract

Seismic safety is an essential consideration in the assessment and redesign of multi-story buildings in Indonesia because structural systems must accommodate earthquake-induced lateral forces while maintaining adequate strength and serviceability. Griya Kos Diponegoro in Semarang is a four-story reinforced-concrete residential building constructed in 2023 and originally designed according to SNI 1726:2012, necessitating reassessment based on the updated SNI 1726:2019 provisions. This study aims to evaluate the seismic response and redesign the upper structural components consisting of reinforced-concrete beams, columns, and slabs. A quantitative structural analysis approach was applied through three-dimensional modeling in ETABS, incorporating response spectrum and equivalent static analyses based on SNI 1726:2019, while reinforced-concrete design followed SNI 2847:2019 and gravity loading followed SNI 1727:2020. The building was modeled using a Special Moment Resisting Frame system on a soft-soil site classified as SE. Modal analysis using 15 modes achieved 100% combined mass participation in both principal horizontal directions, with the first two modes dominated by translational behavior and the third mode by rotational behavior. The design period adopted in the period-check procedure was 0.521 s. Story-drift evaluation indicated that the maximum drift occurred at Floor 2, reaching 29.420 mm in X and 29.348 mm in Y, below the permissible limit of 53.846 mm. The redesign produced member configurations for beams, columns, and slabs that satisfied the evaluated design requirements.

Keywords: *ETABS, Reinforced Concrete, Seismic Redesign, SNI 1726:2019, Special Moment Resisting Frame.*

INTRODUCTION

Earthquake-resistant structural design remains an important engineering requirement in Indonesia because buildings located in seismic regions must accommodate earthquake-induced lateral forces while maintaining structural safety, stability, and occupant protection. The challenge becomes increasingly significant for multi-story reinforced-concrete buildings, where seismic response is governed by the interaction of structural mass, stiffness, strength, ductility, geometry, and the characteristics of the lateral-force-resisting system rather than by gravity loading alone. Indonesia's seismic design framework has continued to evolve through revisions of national standards intended to provide design criteria that are more consistent with contemporary understanding of seismic hazards and structural performance. The current Indonesian seismic provision, SNI 1726:2019, establishes requirements for earthquake-resistant design of buildings and non-building structures and constitutes an important regulatory development from the previous SNI 1726:2012 framework (Badan Standardisasi Nasional, 2019). The determination of site-specific seismic parameters has also become increasingly important because design spectra must reflect the seismic characteristics associated with the geographic location and soil conditions of a structure (Direktorat Jenderal Cipta Karya, 2021). At the broader methodological level, comparative research on earthquake-resistant design standards indicates that differences among seismic provisions can alter design assumptions, seismic demand, and the resulting structural configuration, demonstrating that code selection is not merely an administrative requirement but an analytical determinant of structural design (Yanto, 2025). For residential and other multi-story buildings, the contemporary engineering challenge therefore lies in translating updated seismic hazard information and regulatory provisions into a structurally coherent design capable of satisfying strength and deformation requirements under earthquake loading.

Recent research demonstrates that the implementation of SNI 1726:2019 has become an important component of seismic assessment, analysis, and redesign of reinforced-concrete buildings in Indonesia. Comparative analysis between SNI 1726:2012 and SNI 1726:2019 has shown that revisions to seismic provisions can produce differences in calculated seismic base shear, making regulatory updating potentially consequential for structural analysis and design decisions (Ompi et al., 2025). Studies applying SNI 1726:2019 together with SNI 2847:2019 have demonstrated the practical integration of seismic loading requirements with reinforced-concrete member design through numerical structural modeling using ETABS (Usamah et al., 2022). Research on regular and irregular buildings further indicates that structural configuration affects seismic response, suggesting that the same seismic provisions can generate substantially different structural demands depending on geometry and stiffness distribution (Tarigan et al., 2024). Evidence from a five-story building in Semarang similarly illustrates the relevance of applying the current seismic and reinforced-concrete standards to multi-story structures within the local engineering context (Zhafira et al., 2023). These findings collectively indicate that SNI 1726:2019 provides a necessary regulatory foundation, but its engineering implications must be interpreted in relation to the individual characteristics of the building being evaluated. The literature therefore increasingly shifts attention from simple code compliance toward understanding how updated seismic requirements affect the actual behavior and design adequacy of particular structural systems.

A central limitation in the existing body of research is that seismic redesign cannot be generalized solely from changes in regulatory provisions because every building produces a different combination of modal characteristics, seismic demand, deformation response, and reinforced-concrete member requirements. Studies examining alternative lateral-resisting configurations have shown that changes in structural stiffness and lateral-force-resisting arrangements can significantly influence seismic performance, indicating that structural configuration remains a critical variable in seismic redesign (Kuncoro & Widodo, 2025). Performance-based investigations of reinforced-concrete residential buildings have similarly emphasized that seismic evaluation should extend beyond the calculation of earthquake loads to include the structural response and performance of individual components under the resulting demand (Nurbian et al., 2025). Comparative analysis of reinforced-concrete structures using different spectral approaches has further demonstrated that the selection and interpretation of seismic response parameters can produce different levels of structural demand, particularly when local spectral characteristics are incorporated into the analysis (Sakul et al., 2025). Research on reinforced-concrete structures designed according to SNI 1726:2019 has also employed nonlinear pushover analysis to investigate structural behavior beyond conventional elastic response assessment (Suwondo et al., 2022). At the same time, comparative studies of seismic performance have shown that structural behavior may vary according to building configuration and seismic design assumptions, reinforcing the importance of building-specific evaluation rather than direct transfer of results from one structural case to another (Santana et al., 2026). These findings reveal a conceptual gap between research that evaluates seismic provisions or structural alternatives in general terms and the practical need to reassess a specific existing building whose original design was developed under an earlier seismic standard.

The scientific and practical significance of this reassessment lies in connecting updated seismic demand with the detailed design and verification of the upper structural system, particularly for multi-story residential buildings where structural safety directly affects occupant protection. Seismic assessment requires modal characteristics, mass participation, seismic forces, story drift, and reinforced-concrete member capacity to be interpreted as interconnected components of a single structural system rather than as isolated calculation outputs. Research on equivalent lateral-force procedures under SNI 1726:2019 demonstrates that seismic loading can be systematically evaluated through static-equivalent approaches for multi-story buildings, providing an important basis for structural redesign (Farhany et al., 2023). Recent comparison of ETABS and SAP2000 has further highlighted the computational implementation of equivalent lateral forces under SNI 1726:2019 and the importance of analytical consistency when structural software is used for seismic evaluation (Effani et al., 2026). A resilient-building perspective also emphasizes that structural design decisions should account for the potential consequences of seismic events and should seek configurations capable of maintaining acceptable performance under adverse conditions (Ghimire et al., 2024). The practical problem is consequently not limited to determining whether a building satisfies a revised seismic code, but extends to establishing how the updated seismic requirements translate into appropriate beam, column, and slab dimensions and reinforcement for the investigated structure.

The present study focuses on the seismic redesign of Griya Kos Diponegoro, a four-story residential building in Semarang that was constructed in 2023 and originally designed using SNI 1726:2012. The selection of a four-story residential building is particularly relevant because previous work has demonstrated the feasibility of redesigning comparable four-story structures according to SNI 1726:2019 while highlighting the influence of updated seismic requirements on structural design decisions (Asy'arie & Wicaksono, 2024). The reassessment is conducted using SNI 1726:2019, with the structural system modeled as a Special Moment Resisting Frame and the site condition characterized as soft soil classified as SE. The upper structural system is evaluated through reinforced-concrete beam, column, and slab design, while the lower structure remains outside the scope of the investigation. The structural analysis is performed using ETABS, a computational environment widely employed for numerical structural modeling and seismic analysis of reinforced-concrete buildings (Solusi Konstruksi, n.d.). The analysis considers modal behavior, mass participation, seismic loading, and story drift before translating the resulting structural demands into member-level design requirements based on the applicable reinforced-concrete provisions (Badan Standardisasi Nasional, 2020b). This building-specific framework positions the research between code-based seismic reassessment and practical structural redesign by examining how an actual residential building originally conceived under an earlier standard can be systematically reconsidered using the current Indonesian seismic design framework.

The objective of this study is to evaluate the seismic response of Griya Kos Diponegoro according to SNI 1726:2019 and to redesign its upper reinforced-concrete structural components in accordance with the applicable Indonesian standards. The analysis specifically addresses modal behavior, mass participation, seismic loading, interstory drift, beam design, column design, and slab design as interconnected indicators of structural adequacy. The gravity and seismic loading requirements are incorporated within the structural assessment framework to ensure that the redesign considers both earthquake effects and the minimum loading conditions required for building structures (Badan Standardisasi Nasional, 2020a). The methodological contribution lies in integrating three-dimensional ETABS modeling, response spectrum analysis, equivalent static analysis, and manual reinforced-concrete design checks within a single building-specific redesign procedure. The research also establishes a traceable analytical sequence from site and seismic parameters through global structural response and finally to the required reinforcement configuration of the upper structural members. Rather than proposing a new seismic theory or computational algorithm, the study contributes an applied methodological framework for systematically reassessing a residential building originally designed under an earlier seismic provision and translating the updated requirements into an implementable upper-structure design. The resulting analysis is expected to provide a documented basis for understanding the structural implications of adopting SNI 1726:2019 for Griya Kos Diponegoro while demonstrating how contemporary computational structural analysis can support code-based seismic redesign in an Indonesian urban context.

RESEARCH METHODS

This study employed a non-empirical quantitative structural redesign framework to evaluate the seismic adequacy of the upper structure of Griya Kos Diponegoro in Semarang based on SNI 1726:2019. The investigated building is a four-story reinforced-concrete residential structure with an approximate elevation of 15 m, and the redesign was limited to the upper structural system comprising beams, columns, and slabs. The three-dimensional structural model was developed in ETABS using a Special Moment Resisting Frame system, with building geometry, structural dimensions, material properties, member sections, supports, and structural connectivity defined according to the available design information. Gravity loads were established according to the minimum design loading requirements specified for building structures (Badan Standardisasi Nasional, 2020a). Earthquake-resistant design parameters, seismic loading conditions, and the response spectrum were established according to SNI 1726:2019 (Badan Standardisasi Nasional, 2019). Reinforced-concrete member properties and design requirements were defined according to SNI 2847:2019, while AutoCAD 2025 was used for structural drawings and Microsoft Excel supported independent manual calculations and design checks (Badan Standardisasi Nasional, 2020b).

The technical analysis began with determination of the building risk category, site classification, mapped spectral acceleration parameters, site amplification factors, design spectral parameters, seismic design category, structural response factors, and fundamental period. The resulting seismic parameters were used to construct the design response spectrum and perform modal analysis in ETABS to obtain natural vibration periods, mode shapes, and cumulative mass participation in the principal horizontal directions (Badan Standardisasi Nasional, 2019). Equivalent static analysis was subsequently performed to determine the seismic response coefficient, effective seismic weight, design base shear, and distribution of lateral earthquake forces over the building height, following the application of the equivalent static procedure in SNI 1726:2019-based building analysis (Farhany et al., 2023). Structural performance was evaluated using the fundamental period, cumulative modal mass participation, seismic base shear, and interstory drift, with calculated drift compared against the applicable permissible limit. Internal forces obtained from ETABS were used for the design of representative beams, columns, and slabs, followed by manual checks for flexural strength, shear capacity, reinforcement requirements, confinement, beam-column interaction, and strong-column–weak-beam criteria (Badan Standardisasi Nasional, 2020b). The final redesign was considered acceptable when the calculated structural demands and member capacities satisfied the applicable strength and serviceability requirements, while ETABS outputs and manual calculations were cross-checked to improve analytical consistency and reproducibility.

Figure 1. 3D Structural/Project Model
Source: Author’s Analysis

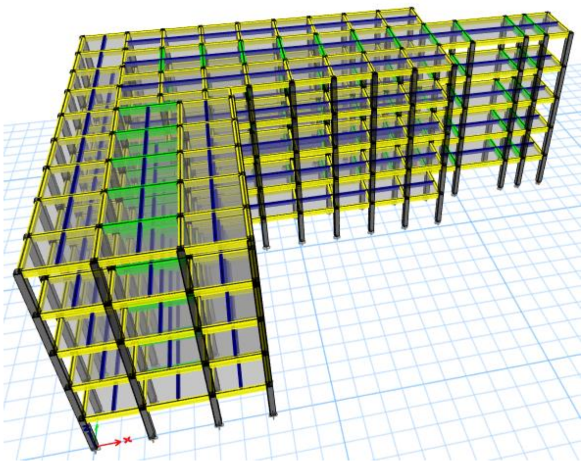

Figure 2. Project Site/Construction Activity Layout
Source: Author’s Analysis

RESULTS AND DISCUSSION

Seismic Parameters, Modal Response, and Story-Drift Evaluation

The seismic evaluation of Griya Kos Diponegoro was initiated by establishing the building classification, site condition, and seismic design parameters required for the structural redesign. The building was classified as Risk Category II with an importance factor (Ie) of 1.0, while the site was categorized as SE, representing soft soil conditions. The mapped spectral acceleration parameters were Ss = 0.8856 and S1 = 0.3788, with site coefficients of Fa = 1.1458 and Fv = 1.9212. These parameters generated Sms = 1.0146 and Sm1 = 0.7277, which resulted in design spectral accelerations of SDS = 0.6764 and SD1 = 0.4852. The relatively high amplification associated with the soft-soil classification indicates that local site conditions contribute substantially to the seismic demand imposed on the structural system (Badan Standardisasi Nasional, 2019). The complete seismic parameters adopted in the redesign are presented in Table 1 as the fundamental input for developing the design response spectrum.

Table 1. Seismic Response Spectrum Parameters

Parameter	Value
Risk Category	II
Importance Factor, Ie	1.0
Site Classification	SE (Soft Soil)
Ss	0.8856
S1	0.3788
Fa	1.1458
Fv	1.9212
Sms	1.0146
Sm1	0.7277
SDS	0.6764
SD1	0.4852
To	0.1435
Ts	0.7173
R	7
Ω0	2.5
Cd	5.5

The resulting characteristic periods were To = 0.1435 s and Ts = 0.7173 s, defining the transition between the short-period and constant-velocity portions of the adopted response spectrum. The structural system was modeled as a Special Moment Resisting Frame with R = 7, Ω0 = 2.5, and Cd = 5.5, representing the force-reduction, overstrength, and displacement-amplification characteristics used in the seismic design procedure. These parameters establish the relationship between the elastic seismic demand and the design-level forces considered in the structural analysis. The resulting seismic design category was D, indicating that the building requires a structural system and detailing provisions appropriate for relatively demanding seismic conditions. The selection of the seismic force-resisting system is particularly important because structural configuration and ductility characteristics directly affect the distribution and reduction of earthquake-induced forces (Farhany et al., 2023). The design response spectrum generated from these parameters is shown in Figure 3.

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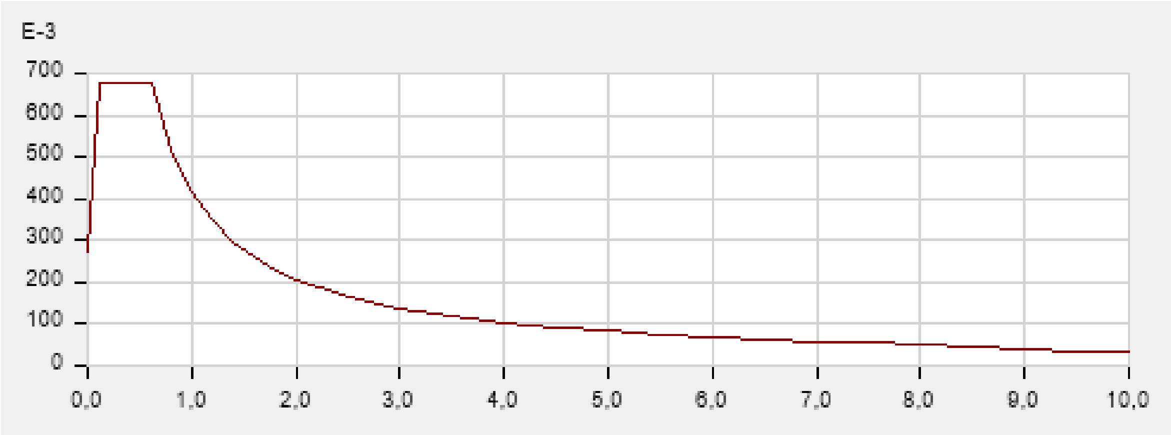

Figure 3. Design Response Spectrum
Source: Author's Analysis

The dynamic characteristics of the building were then evaluated through modal analysis to identify the dominant vibration patterns and establish the period used in the seismic analysis. The calculated lower and upper period limits were 0.3720 s and 0.521 s, respectively, while the first three calculated modal periods were 1.134 s, 1.129 s, and 1.067 s. Modes 1 and 2 represented translational responses in the principal horizontal directions, whereas Mode 3 represented rotational behavior of the structural system. Because the calculated modal periods exceeded the reported upper period limit, the design period adopted in the reported period-check procedure was T = 0.521 s for both principal horizontal directions. This distinction is important because the numerical modal period obtained from the analytical model and the code-controlled period adopted for seismic-force calculation serve different purposes within the design procedure (Badan Standardisasi Nasional, 2019). The detailed dynamic characteristics obtained from the fifteen-mode analysis are presented in Table 2.

Table 2. Modal Period Results

Mode	Period (s)	Frequency (cyc/s)	Circ. Frequency (rad/s)	Eigenvalue
1	1.134	0.882	5.5391	30.6821
2	1.129	0.886	5.5654	30.9738
3	1.067	0.937	5.8885	34.6741
4	0.348	2.872	18.0429	325.5459
5	0.347	2.882	18.1054	327.8063
6	0.327	3.062	19.2365	370.0427
7	0.186	5.386	33.8431	1145.3545
8	0.185	5.394	33.8893	1148.4856
9	0.173	5.773	36.2749	1315.8699
10	0.120	8.310	52.2139	2726.2918
11	0.120	8.318	52.2663	2731.7652
12	0.112	8.968	56.3501	3175.3362
13	0.092	10.889	68.4183	4681.0609
14	0.092	10.918	68.6008	4706.0675
15	0.085	11.832	74.3440	5527.0294

The fifteen-mode modal analysis produced 100% combined mass participation in both the X and Y directions, while translational participation in the UZ direction was 0%. The complete participation in the two principal horizontal directions indicates that the selected modes adequately represent the effective translational mass required for evaluating the earthquake response of the building. The first two modes being predominantly translational also indicate that the principal lateral response is governed by movement along the building's horizontal axes rather than by a dominant torsional mechanism. Meanwhile, the rotational behavior identified in Mode 3 provides an indication that torsional response remains part of the structural dynamic characteristics and should not be disregarded in evaluating the three-dimensional model. Adequate modal mass participation is a fundamental requirement in dynamic seismic analysis because insufficient participating mass may cause the calculated response to inadequately represent the actual dynamic behavior of the structural system (Tarigan et al., 2024). These modal results provide the basis for evaluating the resulting interstory deformation under the adopted seismic loading.

The story-drift evaluation showed that the calculated drift values remained below the permissible limit of 53.846 mm at all investigated stories in both principal directions. The maximum deformation occurred at Floor 2, with drift values of 29.420 mm in the X direction and 29.348 mm in the Y direction, whereas the roof recorded the lowest values of 11.495 mm and 11.314 mm, respectively. Floor 4 and Floor 3 produced X-direction drifts of 19.773 mm and 26.857 mm, while Floor 1 recorded 18.112 mm, demonstrating that the deformation profile varied according to the distribution of structural stiffness and mass along the building height. The small differences between the X- and Y-direction values indicate relatively similar lateral deformation behavior in the two principal directions, with no substantial directional imbalance evident from the calculated drift response. Interstory drift is an important seismic performance parameter because excessive deformation can indicate concentration of lateral demand even when individual structural members remain capable of resisting the applied forces (Nurbian et al., 2025). The complete calculated drift values and acceptance checks are provided in Table 3.

Table 3. Story-Drift Evaluation

Story	X (mm)	Y (mm)	Limit (mm)	Check
Roof	11.495	11.314	53.846	OK
Floor 4	19.773	19.723	53.846	OK
Floor 3	26.857	26.796	53.846	OK
Floor 2	29.420	29.348	53.846	OK
Floor 1	18.112	18.128	53.846	OK

The maximum drift of 29.420 mm in the X direction corresponds to approximately 54.64% of the permissible limit, while the maximum Y-direction drift of 29.348 mm corresponds to approximately 54.55% of the same limit. This margin indicates that the modeled upper structure retains considerable deformation capacity relative to the adopted drift criterion under the analyzed seismic loading condition. The concentration of maximum drift at Floor 2 suggests that lateral deformation is influenced by the stiffness distribution between the lower and middle levels rather than increasing proportionally with building height. The comparable X- and Y-direction responses further indicate that the structural model exhibits a relatively balanced lateral response under the adopted seismic parameters. Similar seismic assessments emphasize that controlling interstory deformation is necessary to ensure that structural response remains within the intended performance range during earthquake loading (Suwondo et al., 2022). The combined seismic-parameter, modal, and drift results establish that the redesigned global structural response satisfies the evaluated deformation criterion and provide the analytical basis for proceeding to the detailed redesign of beams, columns, and slabs.

Reinforced-Concrete Beam and Column Redesign

The global seismic analysis was subsequently transformed into member-level design requirements by using the design forces obtained from the ETABS structural model. The internal forces generated by the seismic and gravity load combinations were used as the basis for determining the required dimensions, longitudinal reinforcement, and transverse reinforcement of the principal reinforced-concrete members. The redesign focused on beams G1, G2, and B1 and column K1 because these members form the primary components of the Special Moment Resisting Frame. Member design was conducted by considering the relationship between the calculated demands and the available flexural, shear, axial, and confinement capacities of the reinforced-concrete sections. This approach ensures that the reinforcement arrangement is derived from the structural response of the complete model rather than from isolated member assumptions (Badan Standardisasi Nasional, 2020b). The resulting dimensions and reinforcement configurations are summarized in Table 4.

Table 4. Reinforced-Concrete Member Reinforcement Summary

No.	Member	Dimensions	Top Support	Bottom Support	Top Span	Bottom Span	Transverse Reinforcement
1	G1 beam	200 × 400 mm	6 D16	6 D16	6 D16	6 D16	4 D10-80 support; 4 D10-150 span
2	G2 beam	250 × 500 mm	8 D18	8 D18	8 D18	8 D18	4 D10-80 support; 4 D10-150 span
3	B1 beam	150 × 300 mm	4 D11	4 D11	2 D11	4 D11	3 D8-100 support; 3 D8-120 span
4	K1 column	400 × 400 mm	—	—	—	—	6 D10-80 support; 6 D10-100 span
5	Slab	120 mm	—	—	—	—	D10-150

Beam G1 was designed with a 200 × 400 mm cross-section and 6 D16 longitudinal bars at both the top and bottom sections for the support and span regions. The identical longitudinal reinforcement at the support and span indicates that the adopted configuration provides a consistent flexural reinforcement arrangement along the member while accommodating the alternating moment demands expected in a moment-resisting frame. Its transverse reinforcement consists of 4 D10-80 at the support region and 4 D10-150 at the span region, producing closer spacing where higher shear and seismic deformation demands are expected. The beam configuration was incorporated into the ETABS model to maintain consistency between the analytical member properties and the reinforcement detailing used in the redesign. The representation of G1 in the structural model is shown in Figure 4, providing a direct illustration of the member configuration used in the analysis.

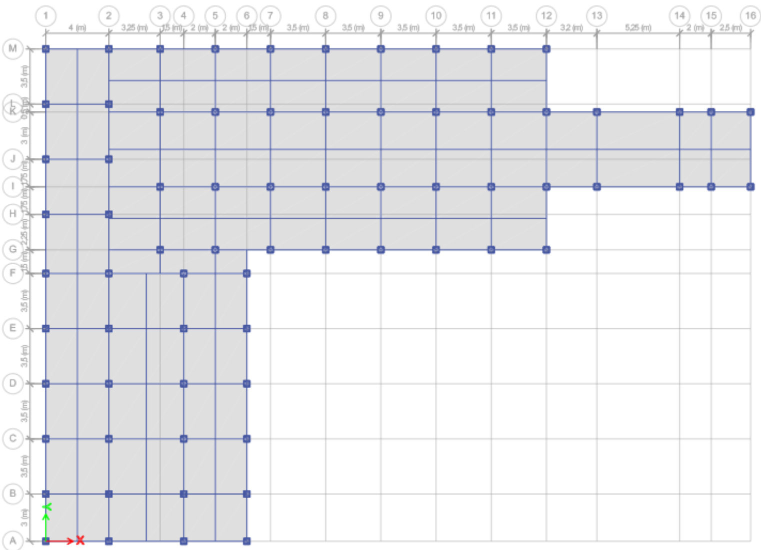

Figure 4. Beam G1 Plan-Sample Modeling
Source: Author's Analysis

Beam G2 required a larger 250 × 500 mm cross-section with 8 D18 longitudinal bars at both the top and bottom sections in the support and span regions. Compared with G1, the greater section dimensions and reinforcement quantity indicate a higher design requirement associated with the internal forces obtained from the structural analysis. The transverse reinforcement of 4 D10-80 at supports and 4 D10-150 at spans follows the same principle of providing closer confinement and shear reinforcement in the regions adjacent to beam-column connections. Beam B1, in contrast, was designed with a 150 × 300 mm cross-section and a different longitudinal reinforcement distribution, consisting of 4 D11 top and bottom bars at supports, 2 D11 top bars and 4 D11 bottom bars at spans. The variation between support and span reinforcement in B1 reflects the different flexural demand patterns along the beam, while its 3 D8-100 support and 3 D8-120 span reinforcement provides additional control of shear

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demand in critical regions (Badan Standardisasi Nasional, 2020b). The reinforcement detailing of the redesigned beams is presented in Figure 5.

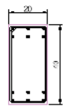
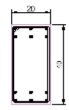
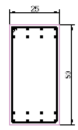
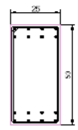
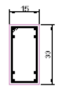
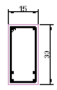
TYPE	TUMPUAN	LAPANGAN	TYPE	TUMPUAN	LAPANGAN	TYPE	TUMPUAN	LAPANGAN
G1 20/40			G2 25/50			B1 15/30		
TULANGAN ATAS	6 D 16	3 D 16	TULANGAN ATAS	8 D 18	8 D 18	TULANGAN ATAS	4 D 11	2 D 11
TULANGAN BAGI	–	–	TULANGAN BAGI	–	–	TULANGAN BAGI	–	–
TULANGAN BAWAH	6 D 16	6 D 16	TULANGAN BAWAH	8 D 18	8 D 18	TULANGAN BAWAH	4 D 11	4 D 11
SENGKANG	Ø 10-80	Ø 10-150	SENGKANG	Ø 10-80	Ø 10-150	SENGKANG	Ø 8-80	Ø 8-120

Figure 5. Beam Reinforcement Detail of Griya Kos Diponegoro
Source: Author’s Analysis

The beam reinforcement configuration demonstrates that seismic member design requires consideration of both longitudinal flexural resistance and transverse reinforcement rather than relying solely on increasing the cross-sectional dimensions. The closer transverse reinforcement provided near the supports of G1, G2, and B1 is particularly relevant because beam ends are potential locations for inelastic deformation under cyclic earthquake loading. The distinction between support and span reinforcement also enables the design to accommodate different moment and shear demands generated along each member. For the Special Moment Resisting Frame, these reinforcement arrangements are intended to support ductile beam behavior and maintain adequate resistance when the structural system is subjected to lateral seismic demands. Proper detailing of beam reinforcement is essential for maintaining the intended plastic mechanism and avoiding premature shear or anchorage failure during strong ground motion (Ghimire et al., 2024). The resulting beam configuration therefore represents the member-level implementation of the global seismic response obtained from the structural analysis.

Column K1 was designed with a 400 × 400 mm cross-section and 20 D24 longitudinal reinforcement bars, accompanied by 6 D10-80 transverse reinforcement at the support region and 6 D10-100 at the span region. The longitudinal reinforcement provides resistance to the combined axial and flexural demands transmitted through the frame, while the transverse reinforcement contributes to confinement and shear resistance under seismic loading. The closer spacing of transverse reinforcement near the support region is consistent with the need to strengthen critical column zones where high combined force and deformation demands may develop. Column design must also be evaluated together with the capacity of the connected beams because an adequate column section alone does not establish the desired seismic failure mechanism. The relationship between column and beam capacities is particularly important for applying the strong-column–weak-beam concept, which seeks to promote beam yielding before significant column failure in moment-resisting frames (Zhafira et al., 2023). The final K1 reinforcement configuration is presented in Figure 6.

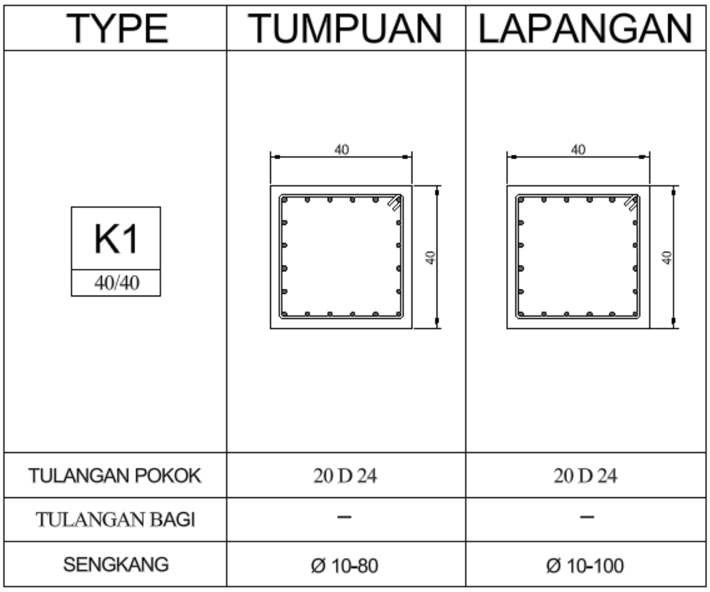


Figure 6. Column Reinforcement Detail of Griya Kos Diponegoro
Source: Author's Analysis

The comparison of the redesigned beam and column configurations indicates that the structural members were proportioned according to their respective demands within the Special Moment Resisting Frame. Beam G1, G2, and B1 use different dimensions and reinforcement quantities, demonstrating that the design forces obtained from ETABS were translated into member-specific reinforcement requirements rather than a uniform detailing scheme. The K1 column incorporates 20 D24 longitudinal bars and closely spaced D10 transverse reinforcement, providing a substantially reinforced vertical member to accommodate the combined axial and lateral demands of the frame. The beam-column relationship was considered through the compatibility of member capacities and the intended strong-column–weak-beam mechanism, while the available numerical results establish the reinforcement configuration adopted for the redesign. This integrated evaluation is necessary because seismic adequacy depends on the interaction between individual member capacity, connection behavior, and the global response of the structural frame (Sakul et al., 2025). The resulting beam and column configuration provides the structural basis for maintaining the intended load-resisting mechanism under the seismic demands established in the global analysis.

Slab Redesign and Integrated Upper-Structure Configuration

The slab redesign was performed as the final component-level stage to complete the configuration of the reinforced-concrete upper structure. The slab was designed with a thickness of 120 mm and reinforcement of D10-150 based on the structural requirements obtained from the overall building model. The adopted slab configuration was treated as a two-way slab, allowing the applied floor loads to be distributed in two perpendicular directions toward the supporting beams. This behavior is relevant to the structural model because the slab does not function independently, but contributes to the transfer of gravity loads and the continuity of the floor system connecting the surrounding structural members. The use of a 120 mm slab thickness with D10-150 reinforcement was evaluated according to the reinforced-concrete design provisions applicable to structural elements (Badan Standardisasi Nasional, 2020b). The two-way slab configuration used in the redesign is presented in Figure 7.

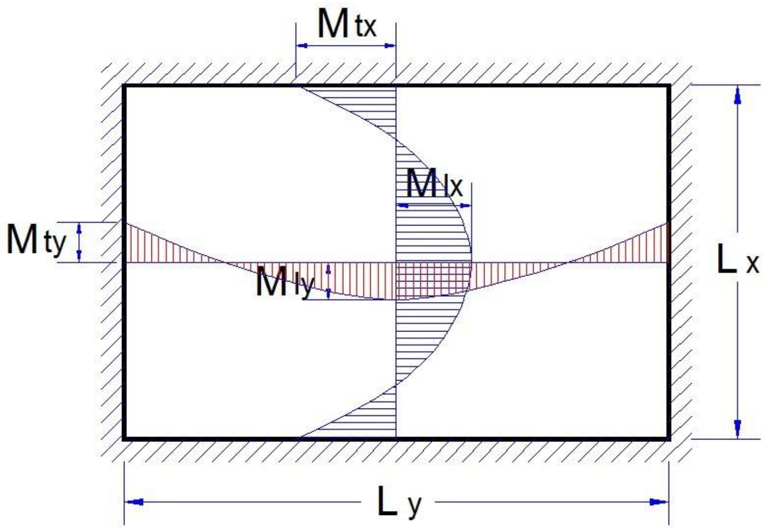


Figure 7. Two-Way Slab
Source: PBI 1971 Table

The two-way behavior of the slab establishes load-transfer paths toward the supporting beams in both principal directions. The 120 mm thickness provides the geometric basis for developing the required flexural resistance, while the D10-150 reinforcement provides distributed reinforcement within the slab panel. The reinforcement spacing also contributes to controlling the distribution of flexural demand and limiting crack development under service loading. Within the upper structure, the slab is supported by beams G1, G2, and B1, so the slab response is directly related to the stiffness and capacity of the surrounding frame members. This interaction is important because changes in slab stiffness and load distribution can influence the internal forces transferred to beams and columns within the structural model (Pratama & ArRasyid, 2026). The resulting reinforcement arrangement is illustrated in Figure 8.

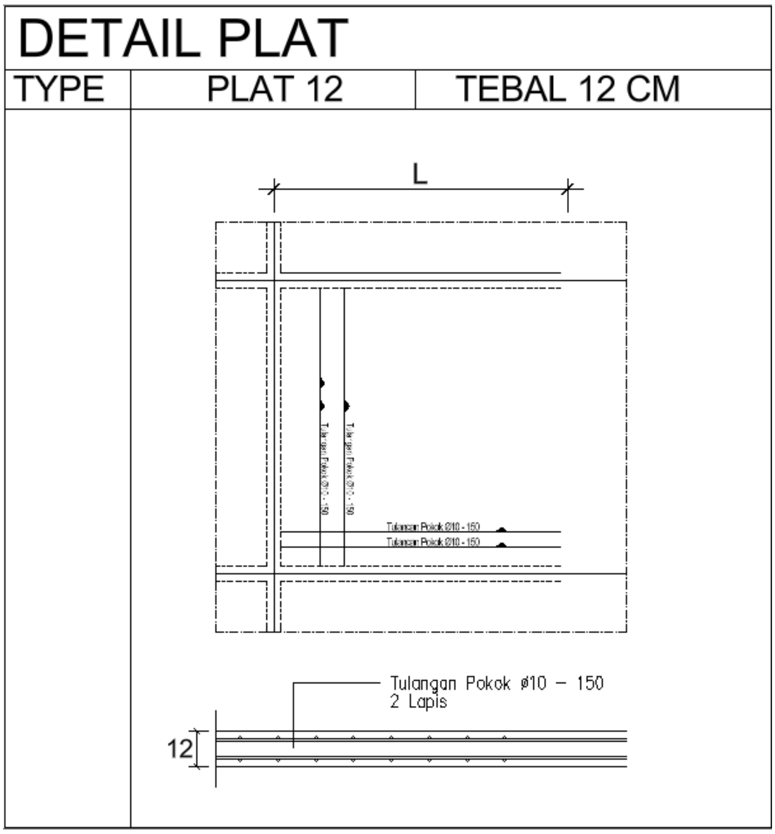


Figure 8. Slab Reinforcement Detail of Griya Kos Diponegoro
Source: Author's Analysis

The slab configuration was then integrated with the redesigned beam and column system to establish a continuous load-transfer mechanism within the upper structure. Gravity loads acting on the slab are transferred through the two-way slab action to the surrounding beams, followed by transmission through the beams to the columns and the primary structural frame. Under seismic loading, the floor system also contributes to transferring lateral forces between the mass of the building and the vertical elements of the Special Moment Resisting Frame. The effectiveness of this mechanism depends on the compatibility of slab geometry, beam dimensions, column dimensions, and structural connectivity within the three-dimensional model. The interaction between floor elements and the primary lateral-load-resisting system is an important consideration because the seismic response of a building is governed by the combined behavior of interconnected structural components rather than individual elements considered separately (Usamah et al., 2022). The final slab reinforcement detail therefore represents an integrated component of the redesigned upper structure rather than an isolated slab calculation.

The consistency of the slab design with SNI 2847:2019 was evaluated through the adopted thickness, reinforcement arrangement, and its compatibility with the reinforced-concrete framing system. The D10-150 reinforcement was maintained as the final slab reinforcement configuration, while the 120 mm thickness was retained as the design section used in the upper-structure configuration. The slab was connected analytically to the surrounding beams and columns so that its contribution to load distribution remained compatible with the structural model used for the seismic analysis. This configuration allows the slab to participate in the structural load path without altering the principal seismic-resisting function of the Special Moment Resisting Frame. Consistency between member dimensions, reinforcement detailing, and structural connectivity is necessary to ensure that the final configuration represents the design assumptions used in the analysis (Suwondo et al., 2022). The resulting slab configuration consequently provides a compatible floor system for the redesigned beams and columns.

The integrated upper structure consists of the 120 mm two-way slab with D10-150 reinforcement, beams G1, G2, and B1 with their respective dimensions and reinforcement arrangements, and K1 columns with a 400 × 400 mm section and 20 D24 longitudinal bars. The combination of these elements establishes a continuous structural system in which floor loads are transferred through the slab and beams toward the columns, while lateral earthquake demands are resisted primarily through the Special Moment Resisting Frame. The differences in beam and column reinforcement are maintained according to the design forces obtained from the structural analysis, allowing each member to respond according to its specific demand and structural role. The integrated configuration also maintains the intended relationship between beam and column capacities so that the seismic-resisting mechanism is not governed prematurely by an undesirable column failure mechanism. Structural redesign based on updated seismic provisions requires this type of system-level consideration because changes in one structural component can affect the demand and response of other interconnected members (Yanto, 2025). The final configuration therefore represents the combined outcome of global seismic analysis and component-level reinforced-concrete design.

The final evaluation indicates that the redesigned upper structure provides a configuration consistent with the seismic design requirements applied to the four-story building. The global analysis produced 100% combined modal mass participation in the X and Y directions, while the maximum story drift of 29.420 mm in the X direction and 29.348 mm in the Y direction remained below the permissible limit of 53.846 mm. At the member level, the redesigned beams, columns, and slab were configured with dimensions and reinforcement corresponding to the structural demands obtained from the analytical model and the applicable reinforced-concrete design provisions. The integration of these components establishes a continuous load path and supports the intended ductile behavior of the Special Moment Resisting Frame under the adopted seismic loading. Application of SNI 1726:2019 to the four-story building therefore has implications beyond the determination of seismic forces, because the resulting demands must be accommodated through compatible member dimensions, reinforcement detailing, drift control, and structural-system behavior (Ompi et al., 2025). The final upper-structure redesign can consequently be interpreted as an integrated seismic-resistant configuration in which the slab, beams, and columns collectively satisfy the evaluated structural requirements and provide a basis for applying the current seismic standard to the building.

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CONCLUSION

The seismic reassessment of Griya Kos Diponegoro based on SNI 1726:2019 demonstrated that the modeled four-story upper structure met the evaluated seismic response requirements. Fifteen modes were included in the modal analysis and produced 100% combined mass participation in both the X and Y directions, with the first two modes dominated by translational behavior and the third mode representing rotational behavior. The design period adopted in the reported period-check procedure was 0.521 s for both principal horizontal directions. The story-drift analysis showed that the maximum drift occurred at Floor 2, reaching 29.420 mm in the X direction and 29.348 mm in the Y direction. Both values remained below the permissible drift limit of 53.846 mm, while all other investigated stories also satisfied the drift criterion. The comparable drift responses in the two principal directions indicate a relatively balanced lateral deformation response of the modeled upper structure. These findings indicate that the redesigned upper structure achieved the evaluated global seismic response criteria under the adopted analytical conditions.

The reinforced-concrete redesign translated the global seismic demands into member configurations for the beams, columns, and slab of the Special Moment Resisting Frame. Beam G1 was designed with a 200 × 400 mm section and 6 D16 longitudinal reinforcement at the top and bottom sections, with 4 D10-80 transverse reinforcement at supports and 4 D10-150 at spans. Beam G2 was designed as 250 × 500 mm with 8 D18 longitudinal reinforcement and 4 D10-80 transverse reinforcement at supports and 4 D10-150 at spans. Beam B1 was designed as 150 × 300 mm with the reported differences in longitudinal and transverse reinforcement between support and span regions. Column K1 was designed with a 400 × 400 mm section, 20 D24 longitudinal reinforcement, and 6 D10-80 transverse reinforcement at supports and 6 D10-100 at spans. The slab was designed with a thickness of 120 mm and D10-150 reinforcement in a two-way configuration integrated with the beam-column system. The combined configuration provides the structural redesign output required to address the seismic demands identified in the analysis while maintaining the intended upper-structure load-transfer and moment-resisting mechanisms.

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