The Modification of the Building Structural Design Using Steel-Concrete Composite and Eccentrically Braced Frames (EBF) System Structure

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Abstract. The construction of high-rise buildings nowadays must be designed to be economical, have a symmetrical and good structure, and must be designed to be earthquake-resistant. Steel structure is one of the earthquake-resistant constructions that are better than concrete structure, since steel structure has a high ductility property. Ductility is the ability of the structure to deform in receiving compressive and tensile forces before the building collapses. EBF can be an alternative in earthquake-resistant buildings since the structure is a combination of two conventional portal systems, where the EBF system has a high level of ductility similar to Moment Resisting Frame (MRF) and also has a high elastic stiffness similar to Concentrically Braced Frames (CBF). The advantage of this system is that it has good structural ductility by designing the shear failure to occur in the link element, while the beam, column, and bracing elements remain in an elastic condition. The EBF Inverted V-type is a good configuration because the largest moment that creates a plastic hinge does not occur near the column, but rather at the ends of the link beam between two bracing joints. Therefore, this model structure is more advantageous because the plastic hinge that occurs does not cause the building to collapse. In addition to the structural system, the use of steel and composite concrete materials in building structures can be a solution in earthquake-resistant building planning. Composite structures are stronger and stiffer than non-composite structures, improving structural performance, allowing for faster construction time, and reducing material costs. Considering the background above, in this study, a modification was carried out on the Field Research Center UGM building which originally used Reinforced Concrete Frame Structure (RCFS) consisting of 3 floors, by modifying it with a composite steel-concrete structure with an EBF system and increasing the number of floors to 25. The EBF configuration used is Inverted V-Braced. This planning refers to and fulfills the structural safety requirements based on SNI 1729–2019, AISC 341–10, SNI 1727–2020, SNI 1727–2013 and SNI 2847–2019, SNI 1729–2020. From the planning results, the floor plate thickness is 11 cm and 12 cm using a material quality of f'c 30 MPa. The main beam dimensions use WF 700x300x15x28. The column dimensions use CFT (Concrete Filled Tube) 800x800x25, CFT 650x650x25, and CFT 600x600x25. The structure is braced using EBF with a link length of 150 cm (Short Link) and bracing dimensions of WF 400x400x20x35. The beam

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dimensions use 400x600 mm. The foundation uses spun piles D60 with a depth of 8 m.

Keywords: building structural design · steel-concrete composite · eccentrically braced frames (EBF)

1 Introduction

Indonesia is a country located in the Ring of Fire region, also known as the Pacific Ring of Fire, which is an area that often experiences earthquakes and volcanic eruptions. One of these areas is Yogyakarta, where there are four active faults that can cause damage to Central Java [4]. High-rise buildings are one of the main targets, and residents living there will feel stronger vibrations. Therefore, it is necessary to construct a building structure that is designed to withstand earthquakes as much as possible to minimize building collapse and prevent loss of life due to earthquakes.

Meanwhile, infrastructure development is rapidly growing, one of which is the development of high-rise buildings. As a tall building, a structure system that is resistant to earthquakes is needed, which is the main thing to consider. Most buildings in Indonesia still use conventional concrete structure systems, which are relatively cheaper and have very low maintenance costs. However, when it comes to earthquake-resistant building structures, steel structures are superior in terms of strength, stiffness, lighter weight, and have high ductility [2, 14]. The steel structure system itself has many types, one of which is the EBF structure system which can be an alternative for earthquake-resistant buildings because the EBF structure system is a combination of two conventional portal systems, where the EBF system has a high level of ductility like the Moment Resisting Frame (MRF) and also has a high elastic stiffness like the Concentrically Braced Frames (CBF) [12].

Apart from the structural system, the use of materials is also important to consider when designing earthquake-resistant buildings. The use of steel and composite concrete in building structures can be a solution in earthquake-resistant building planning [10]. A composite structure is a structure that consists of two or more materials that have different properties and characteristics [6]. In this case, concrete and steel, where concrete has high compressive strength and weak tensile strength, while steel has high tensile strength and high compressive strength as well. Thus, both materials can work together to support the load that acts on a structure optimally. The difference between Reinforced Concrete Frame Structures (RCFS) and composite structures lies in the material that carries the tensile force. In RCFS, the tensile force is carried by reinforcement, whereas in composite materials, the tensile force is carried by steel profile elements [16]. The advantages of composite structures include weight savings in the structure with steel elements, smaller beam profile sections, and increased floor stiffness [7, 13].

By reviewing the aspects mentioned above, this study will conduct a modification planning for the Field Research Center UGM building located in Yogyakarta. The building is constructed on Punukan Street, Kulon Progo Regency, Yogyakarta. The building has 3 floors using Reinforced Concrete Frame Structure (RCFS) system, which will be modified by using a steel composite column structure system with EBF reinforcement in Inverted V-Braced configuration.

The objectives of this journal: (1) To determine the preliminary design of the structural element cross-section of the Field Research Center UGM using an Inverted-V frame (EBF) and link, (2) To plan the loading on the structure of the Field Research Center UGM, (3) To plan the secondary structure, including the slab, secondary beam, lift beam, and stairs, (4) To model and analyze the structure using a software program, (5) To plan the primary structure, including composite main beams and columns, (6) To plan the eccentric braced frames (EBF) and links, (7) To plan the connections that meet the structural planning criteria, (8) To plan the substructure, including the foundation, column footings, and tie beams, (9) To illustrate the results of the structural planning into technical drawings.

This study has problem limitations, namely: (i) The planning is limited to the calculation of the upper structure of the Field Research Center UGM with a height of 25 floors, (ii) Cost and time analysis are not calculated and construction methods are not discussed, (iii) Building planning only reviews the building structure, without considering architectural, mechanical, electrical, plumbing, building utilities, and finishing aspects.

The benefits of having this study are as follows:

1. As input and consideration for the civil engineering world in the planning of steel buildings using Eccentrically Braced Frames.
2. From this planning, it can be known the things that must be considered during planning so that structural failures can be minimized.
3. Adding the author's insights into building planning with steel structures.

There two main components which are very important in this study, namely Composite Structure (CS) and Eccentrically Braced Frame (EBF). A composite structure is a structure that consists of two or more materials with different properties that form a single unit, allowing both materials to work together to optimally support the load acting on a structure. Composite design consists of steel profiles covered with concrete, resulting in a more economical design for bearing compressive or bending loads. The bending load-bearing member is called a composite beam, while the compressive load-bearing member is called a composite column. The advantages of composite structures include weight savings in the structure with steel elements, smaller beam profile sections, and increased floor stiffness [9]. Especially for composite columns, they are an economical solution where a greater additional load capacity is required than using steel columns alone. If one of the loads acting on the column structure is very heavy, the steel size does not need to be increased because additional concrete material is added to resist the load on the column structure. [5].

Whilst, Eccentrically Braced Frames (EBF) is a lateral force resisting steel structural system, which is a combination of two conventional portal systems [8]. EBF has a high level of ductility similar to Moment Resisting Frames (MRF) and also has a high elastic stiffness similar to Concentrically Braced Frames (CBF) [15]. This system dissipates seismic energy by controlling the occurrence of shear yielding, flexural yielding, or a combination of both at the end of each bracing that is connected to a beam so that the force on the bracing is transmitted to other bracings or to the column through shear and moment on the separate segment of the beam called the link beam [11]. The stiffness and ductility of the frame system greatly affect the length of each link. This system limits inelastic behavior to occur only in the link beam between two eccentric braces, while the outer beam, column, and diagonal braces remain elastic during seismic loads [8].

The type of EBF with an Inverted V configuration is the best configuration because the largest moment that creates a plastic hinge does not occur near the column, but rather at the ends of the link beam between two joining braces. Plastic hinges that occur in beams cause beam failure mechanisms, while plastic hinges that occur in columns cause column failure mechanisms, which means building collapse. Therefore, the Inverted V structural model is more advantageous because the plastic hinges that occur do not cause building collapse [1].

2 Methods

This study uses analytical and descriptive methods. In the analytical study, we use a mathematical calculation approach to determine the best value of the models and we use the descriptive method to describe the findings. The analytical and descriptive methods are done to deal with the following: (A) Secondary Structure Design: Floor Roof Slab, Planning of Secondary Beam of Building Floor and Roof Floor, Design of Elevator Support Beam, Design of Staircase; (B) Structural Modeling: Loading, Load Combinations; (C) Design Control: Building Loads Control, Mass Participation Control, Control of Natural Vibration Time, Control of Final Spectrum Response Values, Control of inter-floor displacement; (D) Primary Structure Design: Link Beam, Beams outside link, Bracing, Composite Main Beam, CFT Column; (E) Joint Design: The Connection Between Main Beam and Roof Secondary Beam, The connection between the secondary beam and main beam, Connection Secondary Staircase Beam and Staircase Support Beam, Connection of Stair Landing Support Beam and Column, Connection Between Main Beam and Column, Connection among Column, Bracing Connection, Connection Between Column and Baseplate; (F) Design of Substructure: Building Foundation Design, Pile Cap Design, Pedestal Column Design, Sloof Beam Design.

In general the stages of this study can be seen in the research flow chart depicted in Fig. 1.

The building design which will be modified is displayed in the following data:

Building Name : Field Research Center UGM.

Location: Yogyakarta.

Building Function: Educational Facility.

Number of floor: 25 Floors + 1 Elevator Floor.

Building Height: 111,3 m.

Main Structure: Steel.

Steel Grade: BJ 41.

Concrete Grade: f'c 30 Mpa.

Structural System: Eccentrically Braced Frame (EBF).

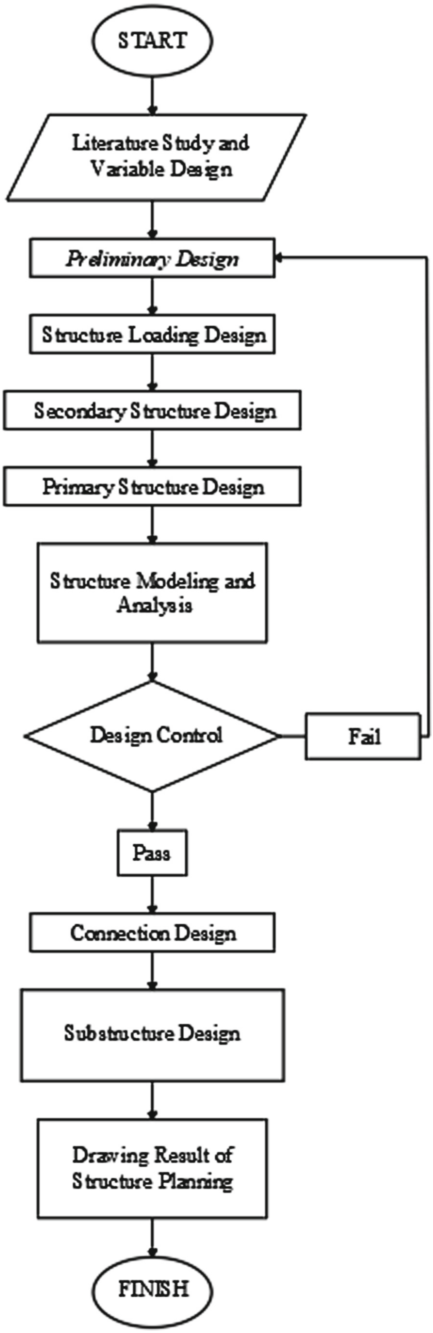


Fig. 1. The research flow chart of this study

3 Research Findings

A. Secondary Structure Design

1) *Floor Roof Slab*

On the ground floor - roof slab, Steel Deck Floor Deck with a thickness of 0.75 mm and a steel tensile strength of 570 N/mm2 is planned to be used. The summary of the floor slab calculation can be seen in Table 1.

2) *Planning of Secondary Beam of Building Floor and Roof Floor*

The secondary beams are planned to be composite steel-concrete beams, using the profiles shown in Table 2.

3) *Design of Elevator Support Beam*

In this building a passenger elevator is used with the following data:

- Elevator Type: Passenger Elevators
- Brand: Hyundai
- Capacity: 15 Orang / 1000 kg
- Clear Opening: 900 mm

Based on the calculation result, a lift supporting beam will be used with the profile of Steel WF 350 x 175 x 7 x 11.

4) *Design of Staircase*

The stairs are planned with the following specifications:

- Floor-to-floor height: 450 cm
- Landing height: 225 cm
- Landing length : 250 cm
- Stair Length: 392 cm

Table 1. Recapitulation of Floor Plate Calculation for Ground Floor [Roof]

Floor	t-Slab (cm)	Reinforcement
Roof	110	M10–200
Floor 1 to 24	120	M10–175

Table 2. Capitulation of Secondary Beam Building Floor and RooF

Floor	Type	Profile
Roof	BA1	WF 400 x 200 x 8 x 13
1–24	BA1	WF 500 x 200 x 9 x 14

- Landing width: 100 cm
- Stair width: 120 cm
- Tread width: 28 cm
- Handrail width: 10 cm
- Riser width: 15 cm
- Muber of treads: 14 set

The calculation result can be seen on Table 3.

B. Structural Modeling

In this structural modeling, a FEM-based software called ETABS 2016 is used as an aid. The structure is modeled as closely as possible to the existing condition so that the results obtained are expected to be close enough. The building structure plan can be seen in Fig. 2 (a).

From the floor plan Fig. 2, the 3D model was created by ETABS, which can be seen in Fig. 2 (b).

1) *Loading*

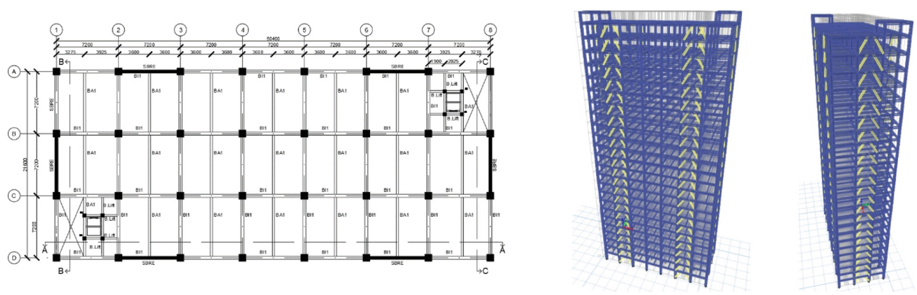
Loading in this building called Field Research Center UGM classified with:

- Dead load

Floor plat : 108 kg/m2

Table 3. Calculation on staircase

Element	Profile
Staircase Slab	Plate 4 mm
Supporting Plate	Siku L 70 x 70 x 6
Landing Slab	Plate 6 mm
Staircase Landing Beam	WF 100 x 50 x 5 x 7
Main Staircase Beam	WF 250 x 125 x 5 x 8
Staircase Support Beam	WF 250 x 125 x 5 x 8



(a). (b)

Fig. 2. (a) Building Structural Plan, (b) The 3D Modeling of Building Structure

Building floor plat: 104 kg/m2

Concrete load: Calculated in ETABS

Steel profile load: Calculated in ETABS

- Live load

Roof Load: 0,96 kN/m2

Floor 1 to 24 (Class): 3,83 kN/m2

Ground Floor: 4,79 kN/m2

- Wind Load

Wind load used SNI 1727: 2013

- Earthquake Load

Earthquake load calculate used "The Procedure for Earthquake Resistant Design SNI 1726:2019".

2) Load Combinations

Referring to SNI 1726–2019 Article 4.2.2, the combination of structural, component, and foundation loads must be designed in such a way that their design strength is equal to or greater than the effects of the factored loads in the combination.

1. $1{,}4D$
2. $1{,}2D + 1{,}6L + 0{,}5\ (L_r \text{ or } R)$
3. $1{,}2D + 1{,}6\ (L_r \text{ or } R) + (L \text{ or } 0{,}5W)$
4. $1{,}2D + 1{,}0W + L + 0{,}5\ (L_r \text{ or } R)$
5. $0{,}9D + 1{,}0W$
6. $1{,}2D + E_v + E_h + L$
7. $0{,}9D - E_v + E_h$

C. Design Control

1) *Building Loads Control*

Weight control aims to limit the software-generated model from deviating too much from the expected result. Therefore, the weight comparison between manual calculation and ETABS 2016 calculation should not exceed 5%. The summary of the calculation can be seen in the following Table 4.

From the calculation results, a difference of 2.19% was obtained which is less than 5%. Thus, the assumption and assumption of the load modeled is in accordance with the planned load.

2) *Mass Participation Control*

Based on SNI 1726:2019, Article 7.9.1, it is stated that dynamic structural response calculations must have a combined mass participation of at least 90% of the actual

Table 4. Summary of Building Loads Control

Load Combination	**Manual (kg)**	**ETABS**
Dead	14797090,84	14918305
Super Dead	289344,84	2676798,72
Live	10223762,96	9707194,492
Total	27914297,64	27302298,21

Table 5. Summary of Mass Participation Control

Modal	Sum UX	Sum UY
…	…	…
10	0,889	0,8824
11	**0,9005**	**0,9003**
12	**0,9045**	**0,9041**

mass in each direction. For a summary of the calculations, please refer to the following Table 5.

3) *Control of Natural Vibration Time*

"To prevent the use of buildings with excessive flexibility, the value of the fundamental natural period (T) of the building structure must be limited. According to SNI 1726–2019, the approximate fundamental period (T_a), in seconds, must be determined from the following equation:

$$T_a = C_t h_n^x = 0,0731 \times 113,5^{0,75} = 2,505 \text{ second}$$

$S_{D1} = 0,60 \geq 0,4$, referring to SNI 1726–2019 Table 17 Assuming that the value of Cu used is 1.4, then the following are the magnitudes:

$$C_u T_a = 1,4 \times 2,505 = 3,506 \text{ second}$$

From ETABS 2016 software, the maximum Ta obtained is 3.075 s. Therefore, it can be concluded that the structure meets the requirements.

4) *Control* of Final Spectrum Response Values

Based on SNI 1726:2019, the final value of dynamic structural response in a certain direction should not be less than 100% of the equivalent static or response value.

- Earthquake in X direction

$$V_{dinamik} \geq V_{statik}$$

$$900986,5\text{kg} \geq 900985,6\text{kg}$$

- Earthquake in Y direction

$$V_{dinamik} \geq V_{statik}$$

$$900986,5\text{kg} \geq 900985,6\text{kg}$$

From the above calculation, the structural modeling still meets the requirements.

5) *Control of inter-floor displacement*

The restriction of inter-floor displacement of a structure aims to prevent non-structural damage and discomfort to occupants. Based on SNI 1726–2019 Ps. 7.9.3, the following are the displacement requirements:

$$\text{The occurred displacement } (\Delta) \leq \text{Allowable displacement } (\Delta a)$$

The summary displacements in every floor can be seen in Table 6.

D. Primary Structure Design

The design of the primary structure includes the design of longitudinal main beams, transverse main beams, links, and connections, including connections between beams, beams and columns, and columns.

1) *Link Beam*

The link beam is designed using WF 700 x 300 x 15 x 28 profile with BJ 41. From ETABS modeling, the link beam is controlled as follows:

$$\begin{aligned} e &= 1,6M_p/V_p = 200,46\text{cm} \\ e &= 150\text{cm} \quad (\text{ Short Link}) \\ V_u &= 31168,7\text{kgfm} < ØV_n = 132030\,\text{kgfm} \\ \alpha &= 0,0141 \text{ radian } < \alpha_{max} = 0,08\,\text{radian} \end{aligned}$$

With $\alpha = 0{,}0141$ rad, braces with a spacing of 50 cm.

2) *Beams outside link*

The beams outside the link are designed using WF 700 x 300 x 15 x 28 with BJ 41, where the required strength of the beam located outside the link is determined based on 1.25 times the nominal shear strength of the link. The interaction between shear and flexure is calculated as follows:

$$\begin{gathered} M_v/ØM_n + 0,625V_v/ØV_n < 1,375 \\ 0,2289 < 1,375 \end{gathered}$$

3) *Bracing*

Bracing is planned to use WF 400 x 400 x 20 x 35. Where axial strength must be controlled by the shear force generated in ETABS 2016.

$$\begin{aligned} P_u \text{ Compression} &= 75681,4\text{kg} \\ P_u \text{ Tensile} &= 46897,4\text{kg} \end{aligned}$$

Table 6. Summary of Drift Displacement

Floor	Drift X (mm)	Drift Y (mm)	Dx (mm)	Dy (mm)	Dmax (mm)	Status
R. Lift	117,959	170,775	0,47	5,03	45	**Passed**
25	117,781	168,889	6,39	10,26	45	**Passed**
24	115,386	165,043	7,58	11,67	45	**Passed**
23	112,544	160,665	8,63	13,07	45	**Passed**
22	109,307	155,765	9,60	14,38	45	**Passed**
21	105,707	150,374	10,50	15,57	45	**Passed**
20	101,771	144,534	11,32	16,66	45	**Passed**
19	97,526	138,287	12,09	17,64	45	**Passed**
18	92,994	131,671	12,79	18,54	45	**Passed**
17	88,197	124,718	13,45	19,36	45	**Passed**
16	83,154	117,458	14,09	20,13	45	**Passed**
15	77,871	109,911	14,27	20,38	45	**Passed**
14	72,518	102,267	14,77	20,96	45	**Passed**
13	66,98	94,407	15,21	21,47	45	**Passed**
12	61,278	86,357	15,57	21,91	45	**Passed**
11	55,438	78,141	15,87	22,27	45	**Passed**
10	49,485	69,79	16,11	22,55	45	**Passed**
9	43,444	61,335	16,29	22,77	45	**Passed**
8	37,337	52,797	16,06	22,54	45	**Passed**
7	31,316	44,343	16,06	22,61	45	**Passed**
6	25,294	35,863	15,91	22,52	45	**Passed**
5	19,326	27,417	15,57	22,14	45	**Passed**
4	13,489	19,114	14,84	21,19	45	**Passed**
3	7,213	11,167	13,22	18,72	45	**Passed**
2	2,965	4,147	7,91	11,06	45	**Passed**
1	0	0	0,00	0,00	45	**Passed**

- Compression Control

$$\varnothing_c \cdot P_n > P_u$$
$$1217362,5\text{kg} > 75681,4\text{kg}$$

- Tensile Control

$$\varnothing_c \cdot P_n > P_u$$
$$1165127,493\text{kg} > 46897,4\text{kg}$$

4) *Composite Main Beam*

The main beams are designed as full steel-concrete composite using WF 700 x 300 x 15 x 28 profile. From ETABS 2016, the beams are controlled as follows:

- Before Composite

$$\begin{aligned} M_u &= 22704\,\text{kgfm} \leq ØM_n = 165240\,\text{kgfm} \\ V_u &= 12025,68\,\text{kgfm} \leq ØV_n = 108621\,\text{kgfm} \\ f &= 0,092\,\text{cm} < f_{ijin} = 720/360 = 2\,\text{cm} \end{aligned}$$

- After Composite

$$\begin{aligned} M_u^+ &= 62837,6\,\text{kgfm} \leq ØM_n = 237315,24\,\text{kgfm} \\ M_u^- &= 103466,3\,\text{kgfm} \leq ØM_n = 165240\,\text{kgfm} \\ V_u &= 41940\,\text{kgfm} \leq ØV_n = 108621\,\text{kgfm} \\ f &= 0,2122\,\text{cm} < f_{ijin} = 2\,\text{cm} \end{aligned}$$

- Shear Connector in Positive Plane

$$\begin{aligned} N &= V_h/Q_n = \tfrac{3029400}{95083,125} = 32\text{ row} \\ S &= L/N = 112,5\text{mm} \sim 65\text{mm} \end{aligned}$$

- Shear Connector in Negative Plane

$$\begin{aligned} N &= V_h/Q_n = \tfrac{353250}{95083,125} = 4\text{ Pieces} \\ S &= L/N = 350\text{mm} \end{aligned}$$

5) *CFT Column*

The column is planned to use a Concrete Filled Tube (CFT) 700 x 700 x 25 with section control as follows:

- Tensile Control:

$$P_p = P_{no} = 3138330\text{kg} > P_u = 1499199\text{kg}$$

- Bending Control

$$M_n = M_p = 422775\,\text{kgfm} > M_u = 42323,6\text{kg}$$

- Initial Interaction Control

$$\begin{aligned} &Pr/Pc + 8/9(Mrx/Mcx + Mry/Mcy) < 1,0 \\ &0,639 \leq 1,0\text{OK!} \end{aligned}$$

- Second Order Axial Strength Control

$$\begin{aligned} &Pr\,/Pc + 8/9(Mrx/Mcx + Mry/Mcy) < 1,0 \\ &0,8962 < 1,0\text{OK!} \end{aligned}$$

- *Strong Column Weak Beam (SCWB) Control*

Nominal Moment of Column

$$\begin{aligned} \mathrm{M_{nc}} &= \mathrm{Z_x} \cdot \left(\mathrm{f_y} - \mathrm{P_u}/\mathrm{A_g}\right) = 425734,454\,\mathrm{kgf} \cdot \mathrm{m} \\ \mathrm{SM_{nc}} &= 2 \times \mathrm{M_{nc}} \qquad\qquad\quad = 2 \times 425734,454\,\mathrm{kg} \cdot \mathrm{m} \\ &\qquad\qquad\qquad\qquad\quad = 851468,9\,\mathrm{kgm} \end{aligned}$$

Nominal Moment of Beam

$$\begin{aligned} \mathrm{SM_{pb}} &= \mathrm{S}\left(1,1.R_y \cdot f_y.Z_x + M_u\right) \\ &= 2 \times ((1,1 \times 1,5 \times 2500 \times 5414) + 62837) \\ &= 668717\,\mathrm{kgm} \end{aligned}$$

$$\mathrm{SM_{nc}}/\mathrm{SM_{pb}} = 1,273 > 1$$

Therefore, it can be concluded that the calculations for columns and beams have met the requirements of Strong Column Weak Beam (SCWB).

For a summary of the primary structure calculations, refer to Table 7:

E. Joint Design

1) *The Connection Between Main Beam and Roof Secondary Beam*

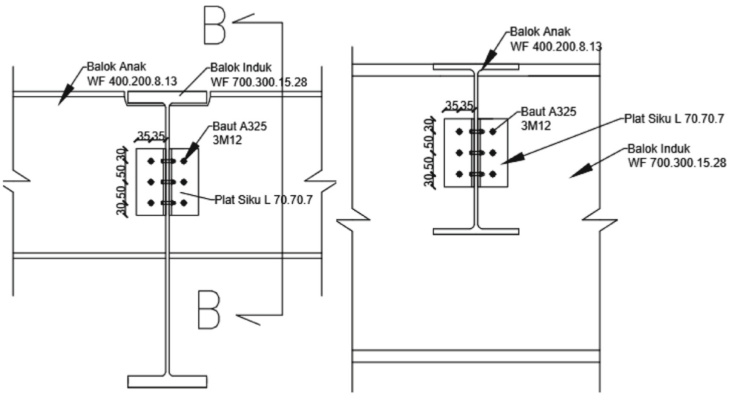
The connection between roof secondary beam and main beam is made using bolts (simple connection), as the roof secondary beam is not designed to participate in resisting earthquake forces. For further details on the connection, please refer to Fig. 3.

2) *The connection between the secondary beam and main beam*

The connection between the secondary beam and main beam is made using bolts (simple connection), as the secondary beam is not designed to participate in resisting earthquake forces. For further details on the connection, please refer to Fig. 4.

Table 7. Summary of Primary Structure Calculations

Element	Floor	Direction	Steel Profile
Main Beam	1- Roof	x	WF 700 x 300 x 15 x 28
		y	WF 700 x 300 x 15 x 28
Column	1–8		CFT 800 x 800 x 25
	9–15		CFT 650 x 650 x 25
	16–25		CFT 600 x 600 x 25
Link	1- Roof	x	WF 700 x 300 x 15 x 28
		y	WF 700 x 300 x 15 x 28
Bracing	1- Roof	x	WF 400 x 400 x 20 x 35
		y	WF 400 x 400 x 20 x 35



(a) (b)

Fig. 3. (a) The Connection Between Main Beam and Roof Secondary Beam, (b) Section B-B of the connection between main beam and roof joist

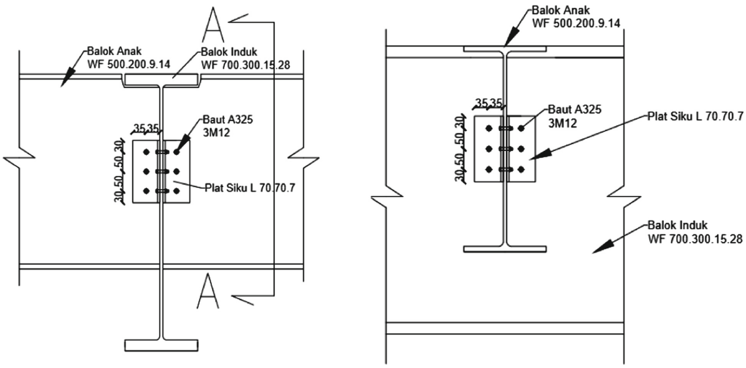


Fig. 4. The Connection Between Main Beam and Secondary Building Beam

3) *Connection of Secondary Staircase Beam and Staircase Support Beam*

The connection of the staircase at the top end is designed as a roller support, so it does not resist lateral forces. Therefore, an M8 A325 bolt connection with a 2.5db = 20 mm long slot hole is installed. For further details on the connection, please refer to Fig. 5.

4) *Connection of Stair Landing Support Beam and Column*

Similar to the connection between secondary beam and main beam, the connection between staircase support beam and column is designed with a pinned connection (simple connection), which only receives lateral loads from the main staircase beam. For further details on the connection, please refer to Fig. 6.

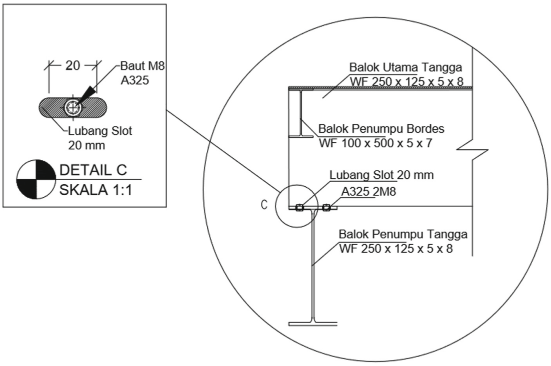


Fig. 5. Connection Between Secondary Staircase Beam and Staircase Support Beam

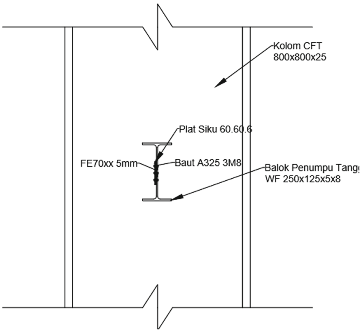


Fig. 6. The Connection Between Staircase Landing Support Beam and Column

5) *Connection Between Main Beam and Column*

The connection between the main beam and column uses a rigid connection type. Therefore, the connection is designed to resist the moment generated by the main beam. For further details on the connection, please refer to Fig. 7.

6) *Connection among Columns*

The columns are connected using welded connections and an additional 50 mm thick plate. For detailed connection information, please refer to Fig. 8.

7) *Bracing Connection*

The bracing connection is calculated using required strength, where it is determined to be greater than or equal to the nominal strength of bracing 1.25 x Ry x Vn. For detailed connection information, please refer to the Fig. 9.

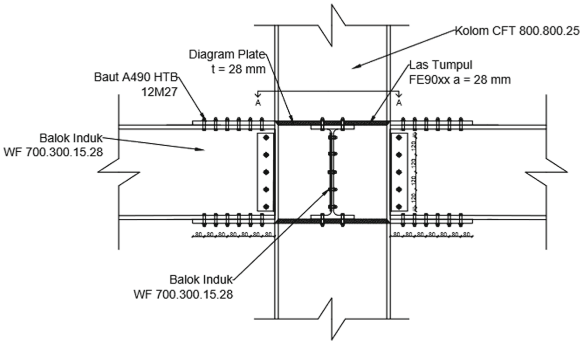
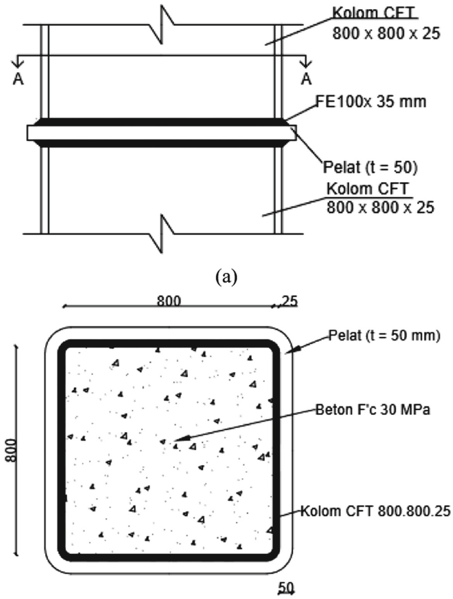


Fig. 7. The Connection Between Main Beam and Column



(b)

Fig. 8. (a) Column Connection Joint (b) Section A-A of Column Connection Join

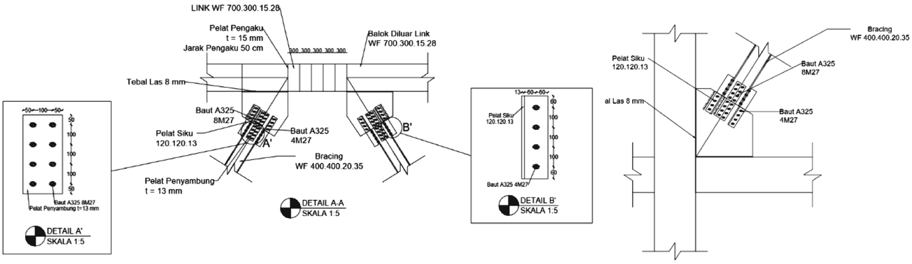
8) *Connection Between Column and Baseplate*

For the connection between the column and the baseplate, a full penetration butt weld is planned around the perimeter of the CFT 800 x 800 x 25. For detailed connection information, please refer to Fig. 10.

F. Design of Substructure

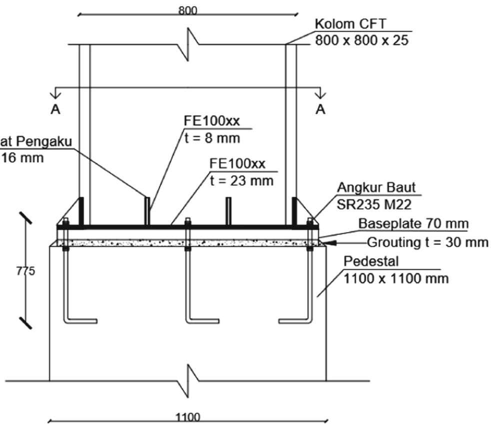
1) Building Foundation Design

The foundation is a substructure element that serves to transfer the loads from the upper structure to the soil on which the building is constructed. The foundation uses PT.

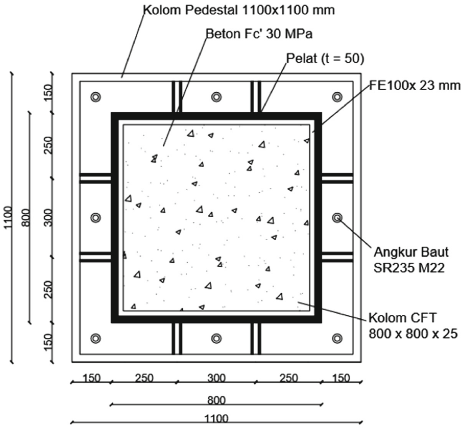


(a) (b)

Fig. 9. (a) Connection between bracing and beam (B) Connection between bracing and column



(a)



(b)

Fig. 10. (a) Column and Baseplate Connection (b) Section A-A of Column and Baseplate Connection.

Wijaya Karya Spun Piles with a diameter of 60 cm and a depth of 30 m. The foundation is controlled as follows:

- Maximum Load Control for 1 Pile
- Material Bearing Control
- Deflection Control
- Maximum Moment Control
- *Pile Cap Design*

Pile Cap or Poer is a part that transfers the load from the column to the pile, so it is necessary to plan and control the loads that occur. The planning of the Poer refers to ACI 318-19M. The Poer is planned as shown in the Fig. 11.

3) *Pedestal Column Design*

The reinforcement on the pedestal column is designed using the SpColumn software. The load that occurs is axial load and moment. The pedestal column is planned with dimensions of 1100 mm x 1100 mm using 16D32 reinforcement evenly on all four sides with D10–150 mm stirrup reinforcement as shown in Fig. 12.

4) *Sloof Beam Design*

The calculation of the beam uses loads that act on the beam including its self-weight, axial compression/tension loads taken as 10% of the column load. The calculation results

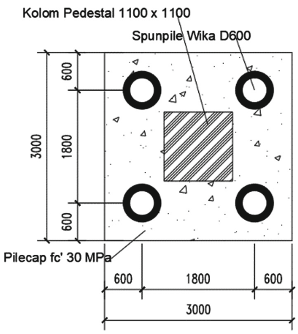


Fig. 11. Pilecap Design

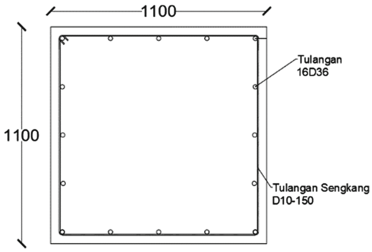


Fig. 12. Tulangan Kolom Pedestal

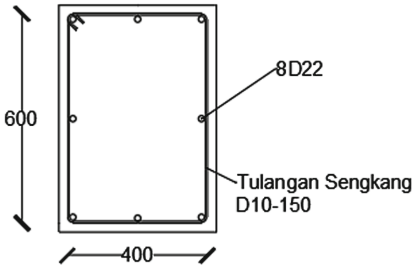


Fig. 13. Sloof Beam Reinforcement

in reinforcement details shown in Fig. 13.

$$\begin{array}{ll} \text{Dimension} & = 400 \text{ x } 600 \text{ mm} \\ \text{Main Reinforcement} & = 8\text{D}22\left(\text{f}_{\text{y}} = 400\,\text{MPa}\right) \\ \text{Shear Reinforcement} & = 2\varnothing 10 - 150 \text{ mm}\left(\text{f}_{\text{y}} = 240\,\text{MPa}\right) \\ \text{Cover} & = 40 \text{ mm} \end{array}$$

4 Conclusion

Based on this research results, we can conclude the following:

The results of the secondary structure calculation have met the requirements for bending control, shear control, and deflection control, which refer to SNI 1729:2015 for steel and SNI 2847:2019 for concrete structures.
The loads used in the calculation are adjusted to the actual location of the building, which is Yogyakarta City.
From the modeling carried out in the ETABS 2016 software, it was found that the structure has met the applicable requirements.
The results of the primary structure calculation and analysis have met the requirements for bending control, shear control, and deflection control. This is followed by checking the rotation angle of the link, bending-shear interaction, and strong column weak beam control.
The connections used are welded and bolted connections.
The results of the lower structure calculation have met the requirements for bending control, shear control, deflection control, and strong column weak beam in accordance with SNI 2847:2019.
The foundation uses Wijaya Karya products with concrete quality of f'c 30 MPa and a diameter of 600 mm. The Pile cap foundation is installed to a depth of 30 m, while the poer foundation uses a thickness of 1.5 m with D25–100 mm and D19–100 mm reinforcement.
The results of the structural analysis are presented in the technical drawing in the appendix.

By this conclusion, we propose the future research, namely the future study can be conducted to evaluate both technical and economic aspects to make the planned structure more effective and efficient. The observation is necessary during implementation in the field to ensure that the structure can perform well and structural failures can be avoided.

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