CERUCUK
Volume 7 No. 8 2023 (394-418)

DESIGN OF THE OFFICE BUILDING OF THE PUBLIC WORKS OFFICE AND SPATIAL PLANNING (PUPR) OF BANJARBARU CITY WITH A STEEL STRUCTURE BASED ON SNI 1729-2020

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ABSTRACT

Steel materials have a superiority in terms of strength, rigidity, and ductility compared to other structural materials, in addition to that with the material being detailed and high *specific strength* can produce a lighter structure. In this study, a redesign of the Banjarbaru City PUPR office building with a steel structure was carried out, this 2 (two) floor building with size of 24 x 36 m^2 was previously designed using a reinforced concrete structure. The designed structure falls into the category of B seismic design and is designed with a common moment-bearing frame system (SRPMB). The design results are for the roof structure used double elbow profiles ⊥50.50.5, ⊥45.45.4, and ⊥30.30.3, for floor plates using floordeck with a thickness of 0.75 with a plate thickness of 110 mm and wire mesh reinforcement M8 – 175. For the child beam and the main beam successively use the WF 250×125×6×9 and WF 300 x 150 x 5.5 x 8 profiles, where the beams are designed as composite beams. While the column uses the H150x 150x10x15 profile. *Rigid Connection* is carried out at the connection of the moment-retaining beams while *simple connection* is carried out at the beam-beam connection. For the foundation used pile foundations with dimensions of 25x25 cm are staked to a depth of 6 m. The results of the structure comparison obtained that the effective weight of the steel structure is almost 50% lighter than the concrete structure, so that the basic shear force, *displacement*, and deviation between floors (*story drift*) that occurs are smaller. For the comparison of the foundation used with the same dimensions and depth in the steel structure, 2 poles are needed, while for the concrete structure it takes 4 pieces to withstand the load of the upper structure.

***Keywords**: Steel Structure, Rigid Connection, Simple Connection. Earthquakes, Composites.*

1. INTRODUCTION

Steel structure is one of the alternative materials used in the construction of buildings and other structures. Steel material is a material that is superior in terms of strength, rigidity, and ductility, this can be seen from its material which is detailed and *high specific strength* so that it can produce a lighter structure. The planning code or specifications used in steel structure planning are always adapted to maintain conformity to scientific developments and market needs. One of the planning methods used in steel structures is

Load and Resistance Factor Design (LRFD), which is a steel structure planning specification issued by AISC (*American Institute of Stell Design*), this planning code has been adopted into a design standard in Indonesia commonly called SNI (Indonesian National Standard) where the term LRFD is absorbed in SNI into Load factor and resistance design (DFBT). For this reason, as material for the design study, a redesign of the structure of the Banjarbaru City PUPR Office Building will be carried out. The 2 (two)-story building with an area of 24 x 36 m^2 which was previously designed using a reinforced concrete structure will be modified using a steel structure with the same structural configuration. The results of the structural design in the form of the upper structure and the lower structure will be compared with the existing conditions of the initial structure that will be recalculated.

2. THEORITICAL STUDY

At the planning stage of the building structure, both from the upper structure to the sub structure must be based on the relevant theories and the Code or standard that applies in this case is the Indonesian National Devil (SNI) so that the results can be accounted for, so that each element of the building structure and the connection of each element reviewed must adjust to the criteria and requirements that have been determined or the calculation method used. This is so that the building structure is ensured to be able to carry the load safely and effectively and can channel the working load to the ground through the foundation.

Referring to SNI 1729:2020 which is a full adoption of ANSI/AISC 360-16, the Design must be made in accordance with the provisions of the Load Factor Design and Durability (DFBK) or with the provisions of the Permit Strength Design (DKI). In this final project in its planning is based on the provisions of the DFBK, the DFBK requirements in SNI 1729:2020 Article B3.3 that the requirements of this specification when the strength of each structural component is equal to or exceeds the specified necessary strength. Based on the combination of DFBK loads. The design should be carried out according to the equation.

$Ru \leq \phi \, Rn$

With:

Ru = Strength needs to use a combination of DFBK loads

Rn = nominal strength

Φ = resistance factor

Φ Rn = design strength

2 METHOD

Planned 2-story building with the following data:

Building name : PUPR Banjarbaru Office

Project Location : Banjarbaru

Building function : Office building

Initial structure of the building : Reinforced concrete structure

Modified Structure : Steel structure

Foundation Plan : Piles

Number of floors : 2 Floors

1) The preparatory stage, which is the stage to find data and information that supports the design of the structure in the form of primary and secondary data that will be used in the redesign of the Office Building of the Public Works and Spatial Planning Office (PUPR) of Banjarbaru City,
2) *Preliminary Design*, which is the stage in estimating the initial dimensions of the structural elements, determining the quality of the material or material of the structure and planning the dimensions of the profile to be used,
3) The calculation of loading includes dead load (D), live load (L), wind load (W) and other load combinations in accordance with SNI 1727:2020. As for the calculation of earthquake load (E) refers to SNI-1726-2019,
4) The planning of the secondary structure will be carried out in advance before the planning of the main structure because the secondary structure will pass the existing load to the main structure. The secondary structure that will be planned in this final project is as follows:
   a. Roof structure
   b. Floor plate
   c. Joist
   d. Staircase structure

5) Creation of 3D modeling of structures with the application of ETABS v17, creation of models based on architectural drawings obtained from the results of data collection. Then input the calculated load data and the load combination used in the ETABS v17 program. Continued with structural analysis to determine the structural response in the form of inner force and deflection that occurs in structural elements
6) The results of the structure analysis must be controlled against certain limitations to determine the feasibility of the structural system. The things that must be controlled are as follows:
   a. Control of mass participation
   b. Fundamental natural vibrating time control
   c. Control the final value of the spectrum response
   d. Control of deviation limits (drift)

   From this analysis, it was seen that the structural response was in the form of a deep force that occurred in each element of the structure to check the cross-sectional capacity based on the planning code used. If the structural design does not meet the specifications and safety, it must be checked again for the preliminary design and loading stages of the structure. If the design of the structure has met the regulations used in planning, the results can be used in planning,
7) Connection calculation based on applicable specifications,
8) Foundation calculation, on foundation planning based on the results of the Cone Penetration Test (CPT) test, to calculate the carrying capacity of the foundation.
9) Comparing the steel structure design results against the existing concrete structure that has been recalculated.

3 RESULTS AND DISCUSSION

1. Secondary Structure Planning

Secondary structure planning includes roof structures, floor slabs, child beams, and stairs

A. *Roof Structure Planning*

The following is the following data on the planning of the easel:

Horseshoes: 18 m

Distance between horses: 3 m

Distance between purlin: 1.5 m

Roof angle: 31.29°

Steel roof height: 547 cm

Roof construction : Rigid Frame

Roof covering of the building : Galvalume

Connection : Bolts

Details of the roof structure can be seen in Figure 1.

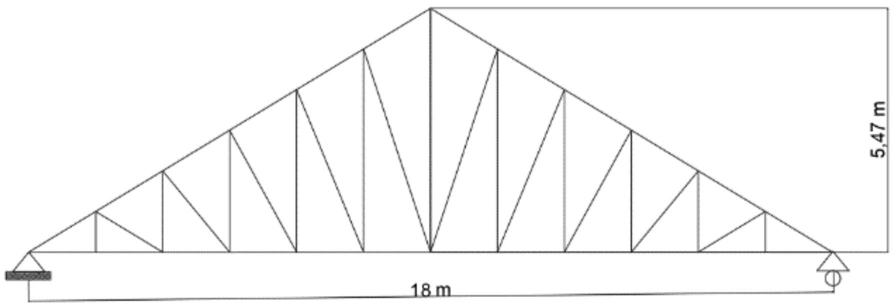


Figure 1

The results of the design of the easel can be seen in Table 1

Table 1 Roof Structure Planning

Element	Profile
Purlin	C 100 50
plates	10 mm plate
Inner diagonal bar	2L 45 x 45 x 4
Outer diagonal bar	2L 50 x 50 x 5
Vertical bars	2L 45 x 45 x 4
Horizontal bars	2L 30 x 30 x 3

B. *Floor slab Planning*

In this floor plate planning, it will use floordeck from the Super Floor Deck which functions as positive reinforcement while for negative reinforcement it will use *wire mesh*. The calculation results can be seen in Table 2

Table 2 Floor slab planning

Plate	Span (m)	Plate Thickness (cm)	*Wire mesh*
Floor	6	11	M8-175

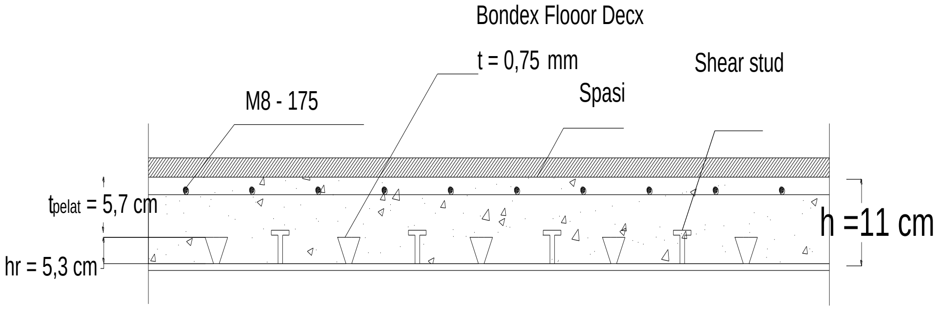


Figure 2 Floor Plate Looping

C. Children's Beam Planning

In the planning of the child beam using a WF profile of 250×125 ×6× 9, where the child beam is designed composite with sliding connectors d = 16 mm, Fu = 450 MPa, used 2 x 13 sliding links with a distance of s = 450 mm.

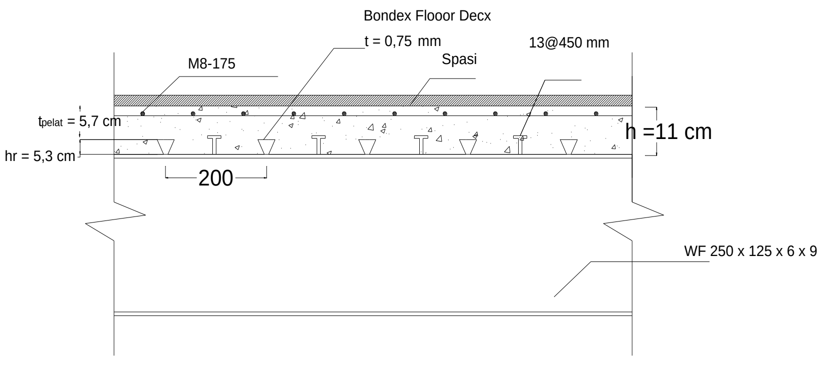


Figure 3 Children's Beam

D. Ladder Planning

Steel quality: BJ 37

Height between floors: 430 cm

Bordes height: 215 cm

Stomping height (t) : 20 cm

Stomping width (i) : 25 cm

Number of climbs (Σt) : 11 pieces

Bordes width: 145 cm

Bordes length: 120 cm

Ladder length: 275 cm

Ladder width: 120 cm

Angle of inclination (α): 38,038 ^{o}

Table 2 Ladder Planning

Stair Elements	Profiles used
Treadle plates	4 mm plates
Treadle plate support	L50 x 50 x 6
Plate Edges	8 mm plates
Bordes Beam	I 100 x 100 x 6 x 8
Stair Beams	WF 175 x 90 x 5 x 8

2. Main Structure Planning

A. *Structural Modeling and Analysis*

Modeling the upper structure in this final project uses moment *frame systems* (SRM). This SRM structural system functions as a buffer for lateral forces that occur due to earthquakes.

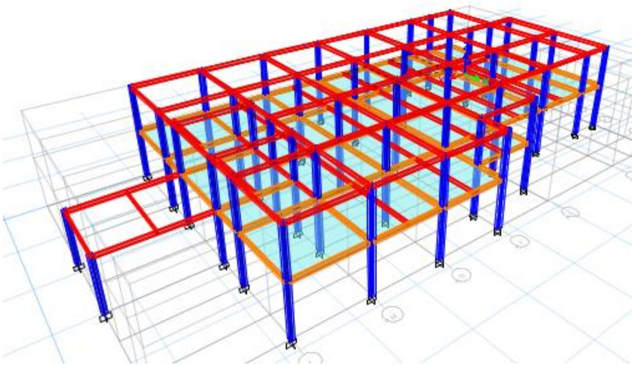


Figure 4. Steel Structure Modeling

Modeling designed as a moment-bearing system are frames in which the beam wings must be spliced to the wings of the columns to be modeled as clasp (rigid) joints while the portals that do not meet the above conditions will be used as a portal to withstand the force of gravity where in the modeling it will be sparated.

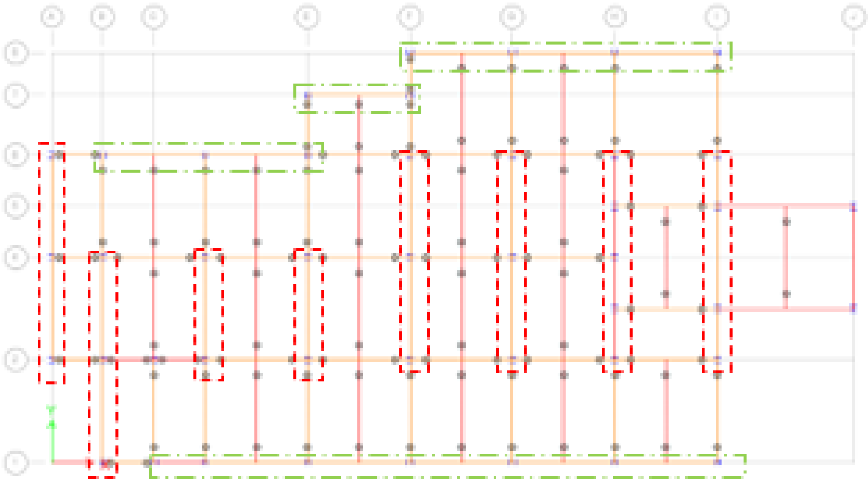


Figure 5. Moment Bearer Portal

where is the X direction moment holding frame (green dotted line) which serves to withstand the earthquake force from the X direction, and the Y direction moment holding frame system (red dotted line) to withstand the earthquake force from the Y direction.

a. Control Modeling structure

To ensure structural modeling matches the actual modeling requires manual checking of any of the columns reviewed with a 1D+ combination! L.

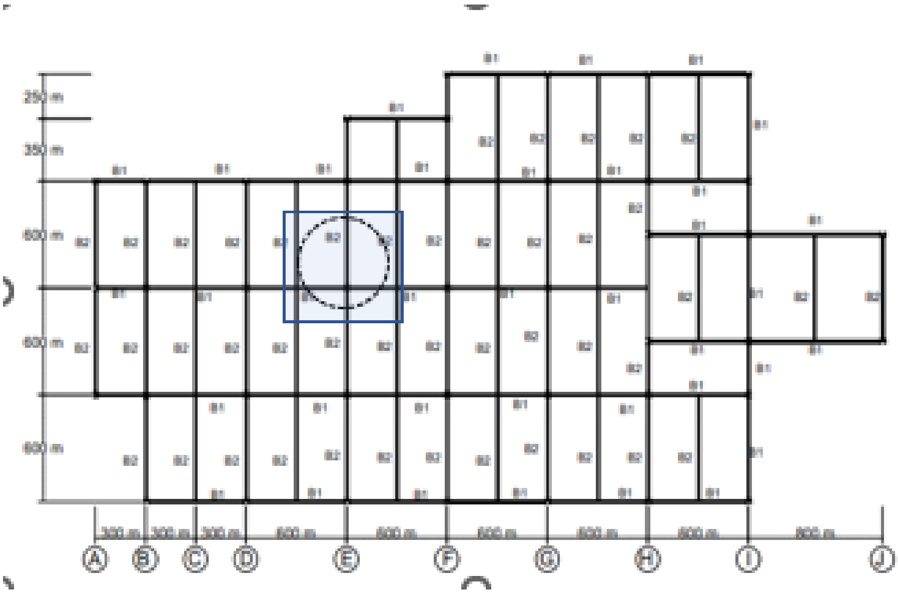


Figure 6. Trucking Plan

The calculation difference between the manual calculation and the Etabs result is as follows:

Manual Result: 21018,48 kg

Application result: 21041.41kg

21041,41kg – 21018.48 kg = 21.93 kg

So that the percentage is obtained:

$$selisih\ (\%) = \frac{21{,}93}{20595{,}59} \times 100\% = 0{,}104\ \%$$

b. Control of Mass Participation

Based on SNI 1726-2019 Article 7.9.1.1, the Analysis must include a sufficient number of varieties to obtain the participation of the combined variety mass of 100% of the mass of the structure.

Table 3 Control of Mass Participation

Case	Step type	Step Num	Sum UX	Sum UY
Text	Text	Unitless	Unitless	Unitless
Capital	Mode	19	1	0,999
Capital	Mode	20	1	1

c. Natural Vibrate Time Control

the fundamental period of the structure should be determined from:

$T_a = Ct.h_n^x; Ct = 0{,}0724$; x $= 0{,}8$; hn $= 8{,}1$m

$$T_a = Ct.h_n^x = 0{,}0724\ \times 8,1^{0,8} = 0{,}386\ s$$

With the value of S_{D1}= 0.087, then obtained Cu = 1.7

$T_{mak} = Cu.T_a = 1{,}7 \times 0{,}386 = 0{,}656\ s$

Table 4 Structure Periods

Case	**Mode**	**Period** sec	**Frequency** cyc/sec
Capital	1	0,645	1,551
Capital	2	0,591	1,692
Capital	3	0,498	2,009

T = 0,654 < T_{mak}=0,656 ..... OK

d. Control of Base Shear

Based on SNI 1726-2019 Article 7.9.1.4.1, if the combination of responses for the basic shear force the result of the variance analysis (V_t) is less than 100% of the shear force (V) calculated through the static equivalent method.

Table 5 Base *shear* control

KET	V dynamic (KN)	V Statics (KN)	KET
RSX	166,597	188,18	Not OK
RSY	156,415		Not Ok

Since the basic shear forces of the X and Y axis directions do not meet the requirements *of* $V_t > V$, the shear forces of the X and Y axis directional response spectrum need to be corrected as follows:

Initial FS value $= \frac{g\ x\ Ie}{R} = \frac{9.8147 \times 1}{3,5} = 2,802\ m/s^2$

FSX = $2,802 \times \frac{V}{V_{tx}} = 2,802 \times \frac{188,18}{166,597} = 3,165\ m/s^2$

FSY = $2,802 \times \frac{V}{V_{tY}} = 2,802 \times \frac{188,18}{156,416} = 3,371\ m/s^2$

Table 6 Corrected *control base shear*

KET	V dynamic (KN)	V Statics (kN)	KET
RSX	188,18	188,18	OK
RSY	188,18		OK

e. Story Drift Control

Story Drift deviation refers to SNI 1726-2019 Article 7.12.1

Table 7 Deviations between floors

Story	***Displacement*** δe_X **(mm)**	δe_Y **(mm)**	***Elastic Drift*** δe_X **(mm)**	δe_Y **(mm)**	***h*** **(mm)**	***Inelastic Drift*** Δ_X **(mm)**	Δ_Y **(mm)**	***Drift Limit*** Δ_a **(mm)**	***Cek***
2	0,1022	0,049	0,033	0,026	3800	0,100	0,078	76,000	OK
1	0,069	0,023	0,069	0,023	4300	0,207	0,069	86,000	OK

B. *Main Beam Planning*

In the planning of the children's beams using a WF profile of 250×125 ×6× 9, where the child beams are designed compositely with sliding links d = 16 mm, Fu = 450 MPa. Obtained results

Conditions before Composite

With scaffolding in the middle of the span is obtained:

φMn = kgm > 9300,20Mu = 6363 kgm

φVn =23601 kgm > Vu = 3612.37 kg

f = 0,907 mm < *physical* = 8,3 mm

Conditions after Composite

φMn^{+} =21594,564 kgm > Mu^{+} = 19901.56 kgm

φMn^{-} =14396.22 kgm > Mu^{-} = 5366.34 kgm

φVn =23601 kgm > Vu = 6345.62 kg

f = 0,794 cm < *physical* = 1,67 cm

Installed 2 sliding connectors in one row.

On the area of positive moments; N = 7 pieces

In the area of negative moments, N = 2 pieces

Distance between sliding connectors, S = 35 cm

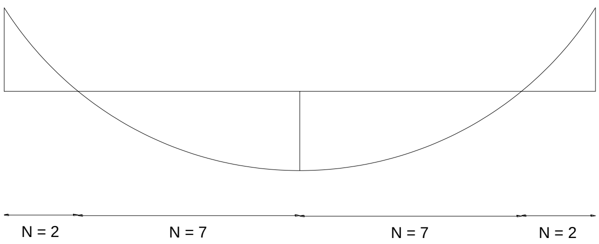


Figure 7. Sliding Link Placement

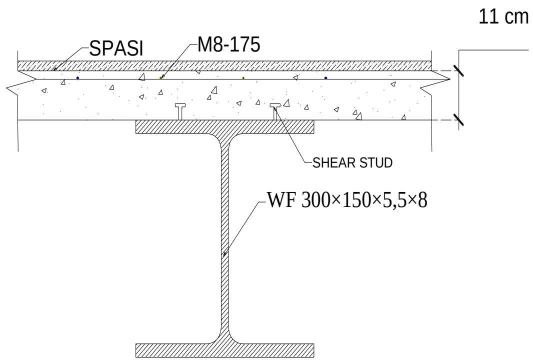


Figure 8. Composite Beams

C. *Main Column Planning*

In the planning of the child beams use the H profile 300×300 ×10×15.

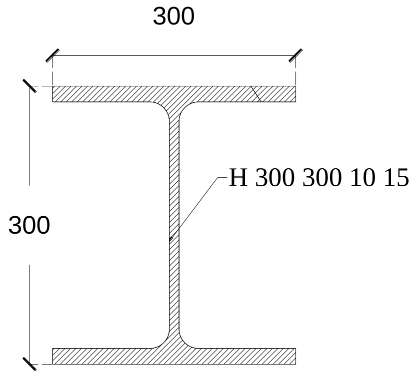


Figure 9. Column H 300 x 300 x 10 x 15

$\frac{Pu}{\phi Pn} < 0{,}2$; So

$$\frac{Pu}{2.\phi Pn} + \frac{8}{9}\left(\frac{M_{ux}}{\phi M_{nx}} + \frac{M_{uy}}{\phi M_{ny}}\right) \leq 1$$

$0{,}678 \; < \; 1 \, .. \, \mathrm{OK}$

3. Connection Planning

For connection planning as follows:

A. Rigid Connection

Beam and column joints are designed as rigid joints

Column: H 300 300 10 15

Beam: WF 300×150×5.5×8

Diameter builds: Ø16 mm

Bolt type: A325

Fnv: 620 MPa

Fnt: 372 MPa

Connecting Profile: L100 x 100 x 10

Connecting Profile: T200 x 200 x 6.5 x 9

Plate Quality: BJ 37

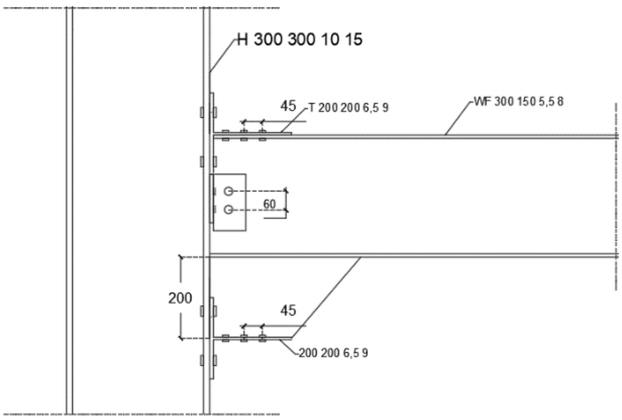


Figure 10. Bolt Connection Main beam and Column

B. *Simple* Connection

Joint planning with A325 quality bolt type, spliced with elbow profile with BJ37 quality

Table 8 Simple connections

Element		Profile L	D (mm)	n (fruit)
Child beam -	Main Beam	L 40 40 4	16	2
Main Beam Staircase-	Stair Stacking Beam	L 40 40 4	16	2
Main Beam of stairs-	column	L 40 40 4	16	2

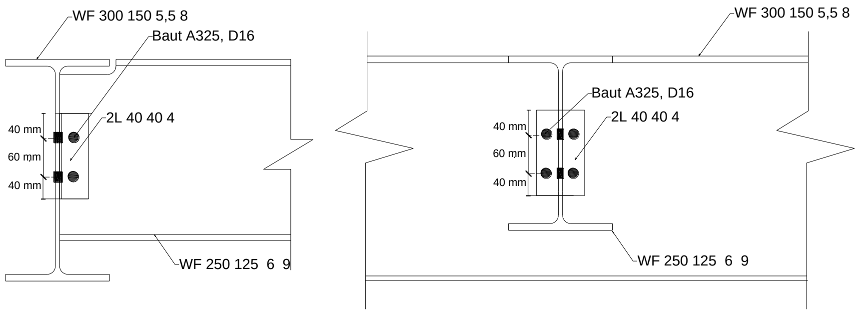


Figure 11. Joints of the Main Beam and Child beams

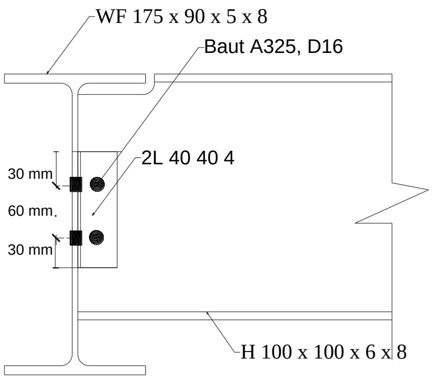


Figure 12. Joints of the Main Beams of the Ladder and the Stacking Beams of the Stairs

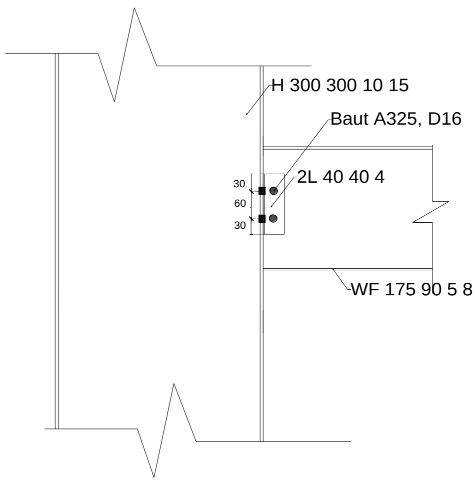


Figure 13. Beams of children and ladders

C. Base Plate Connection

Renovated base plate with dimensions of 40 x 40 cm with quality BJ 37 with a thickness of 2 cm, and installed 4 pieces of D16 transport with quality BJ37 which are planted at a depth of 20 cm

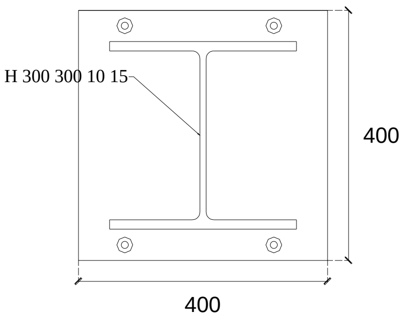


Figure 14. *Base Plate*

4. Bottom Structure Planning

concrete piles measuring 25 x 25 cm products from PT WIKA are used. Designed to a depth of 6 m, the carrying capacity is produced based on Cone Penetrometer Test obtained the carrying capacity of one Qizin pole = 135 KN, and 2 piles are needed. For the maximum load of group poles is obtained on the basis of the following calculations:

$$P_{mak} = \frac{P}{N} \pm \frac{My.X}{\sum X^2} \pm \frac{Mx.Y}{\sum Y^2}$$

$$= \frac{206{,}04}{2} + \frac{28{,}7.0{,}45}{0{,}405} + \frac{27{,}136.0}{0} = 134{,}91\ kN$$

$P_{mak} = 134{,}91\ kN < Qizin = 135{,}66\ kN$... OK

Deflections are obtained as well as moments along the posts based on lateral forces acting as follows:

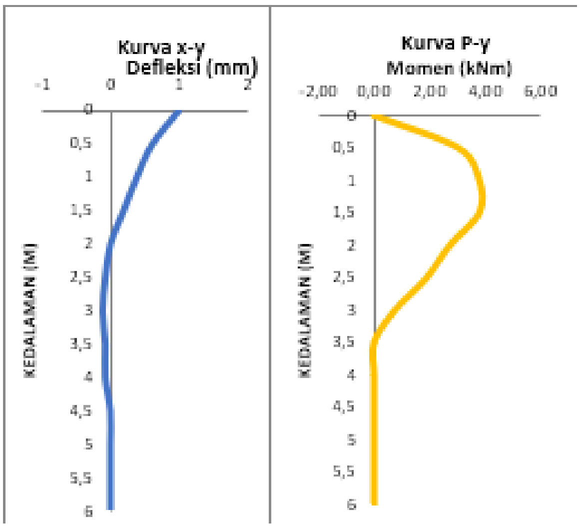


Figure 15. Deflections and moments along the post

$M_{mak} = 9{,}99\ kNm < M_{crack} = 29{,}9\ kNm$ ..... OK

$x = 1{,}8007\ mm < x_{izin} = 12\ mm$ .... OK

As well as based on the results of the calculation of the decrease obtained Δs_c= 0.73 cm < sizin = 15 cm

Pile cap planning with dimensions of 1.6 x 0.7 m based on the calculation results used D16-180 with temperature reinforcement D16 – 300 can be seen in Figure 16

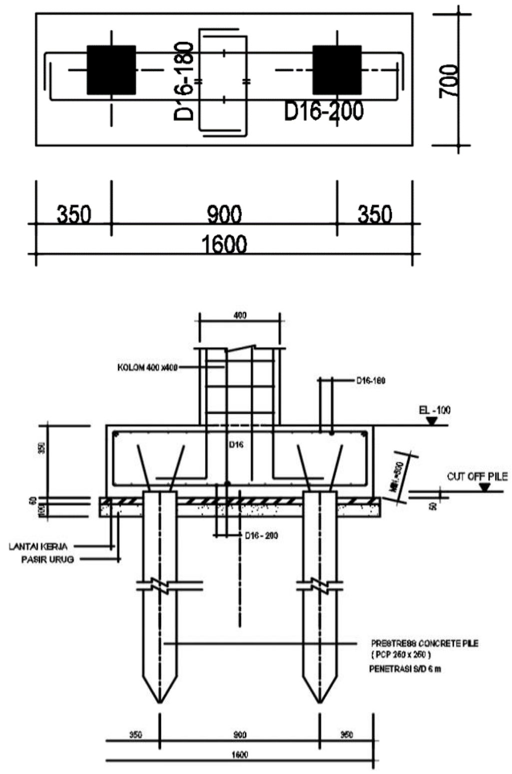


Figure 16. Pile Cap Repeating

For the planning of pedestal columns with a length of 1 m, dimensions of 40/40 cm were obtained with concrete quality K-275, as well as the quality of the main reinforcement *Fy* = 400 MPa, and the quality of the rebar *Fys* = 240 MPa. Used main reinforcement 8-D16 and sliding reinforcement φ10-250.

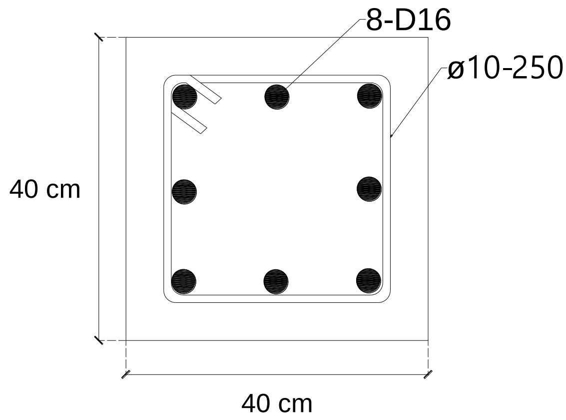


Figure 17. Pedestal column Cross-section Details

Meanwhile, in the sloof planning , dimensions of 20/30 cm are used, the main reinforcement is used 6-D22 and the zinc reinforcement φ10-200

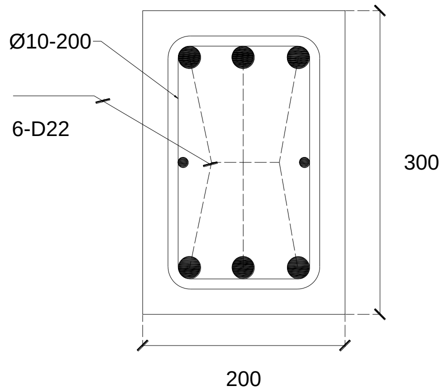


Figure 18. Sloof beam Details

5. Structure Benchmarking

As a comparison of buildings that were previously designed using reinforced concrete structures with new buildings that have been modified using steel structures, it will be compared practically regarding the effect of the structural system on the working earthquake load, the comparison of beam elements, columns, and the comparison of the foundations used.

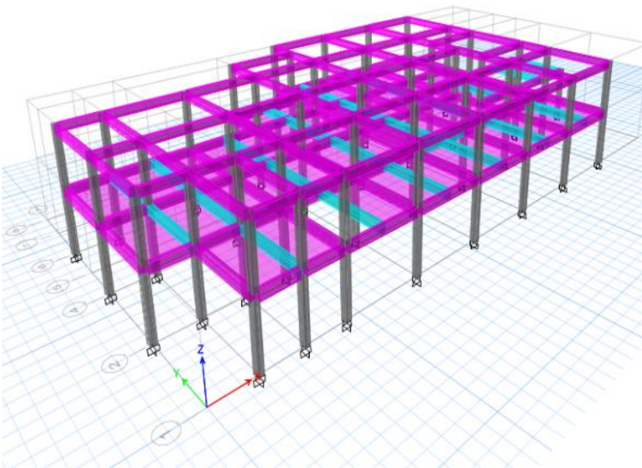


Figure 19. Re-modeling of existing structures

A. Comparison of Earthquake Forces and Their Effects on Structure

For comparison of the calculation of the comparison of basic shear forces from the results of the analysis of a combination of variations that have been corrected with the value of the equivalent static base shear force, the shear force due to the seismic load on the steel structure is smaller than the shear force that occurs in the concrete structure. This can be seen from the shear force of the variety analysis results that have been corrected on the concrete structure of 404,311 KN, where the result is almost twice as large as the shear force of the variety analysis results that have been corrected in the steel structure by 188,180 KN, this is because the magnitude of the basic shear force in the structure is greatly influenced by the magnitude of the effective weight of the structure, In Figure 20 , it can be seen that the weight of the steel structure is almost half times lighter than the structure using reinforced concrete.

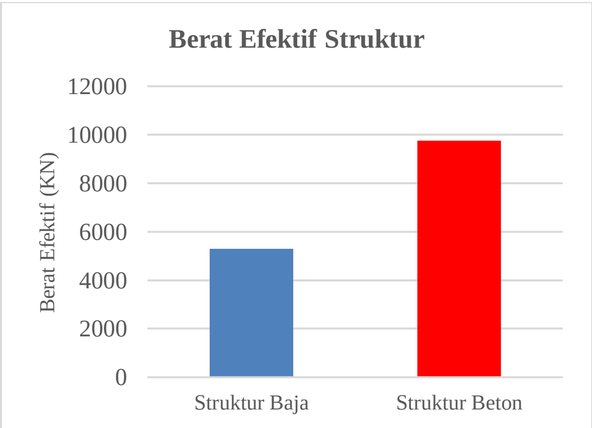


Figure 20. Comparison of the effective weight of the structure

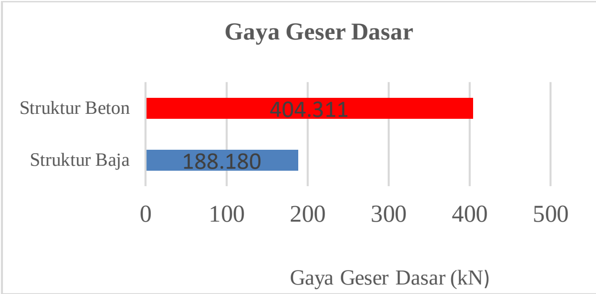


Figure 21. comparison of basic shear forces

From the calculation results, it shows that the modified structure has undergone a smaller diplacement than the real structure, and the deviation between floors in the modified structure shows a smaller value than the real structure, but from both structures it is still within the deviation limit of the permit hinted at by SNI 1726-2019.

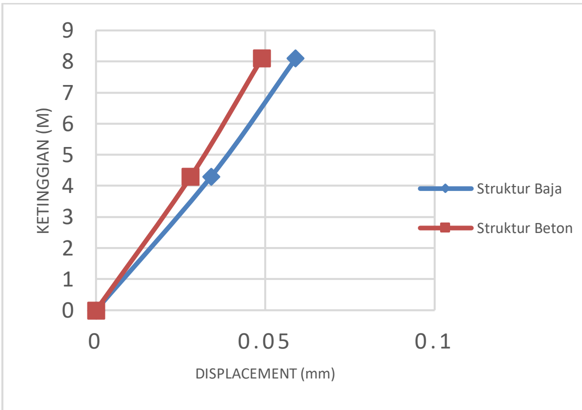


Figure 22. *diplacment*

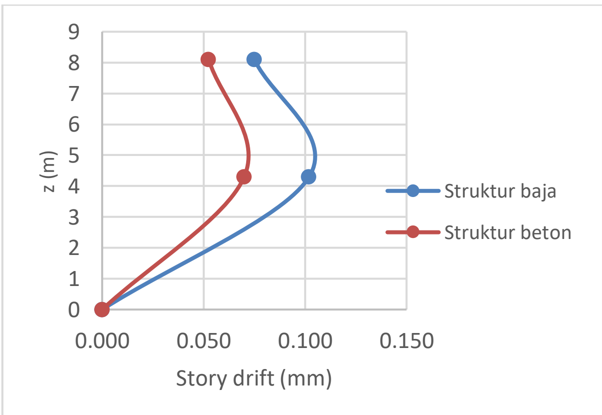
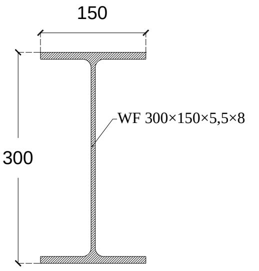
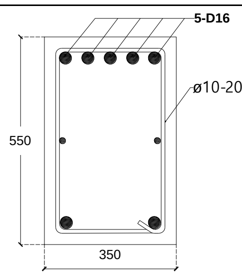
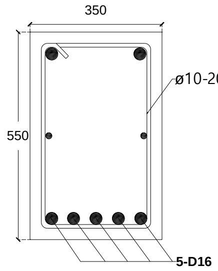
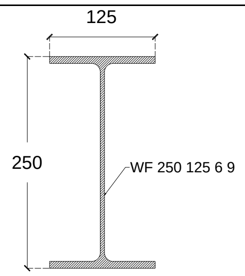
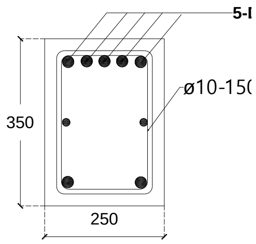
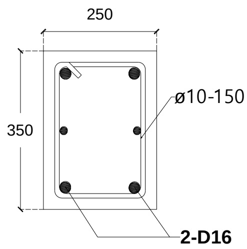


Figure 23. Comparison of *Story Drift*

B. Beam Comparison

For comparison of beams on modified structures and existing structures dpata is seen as follows

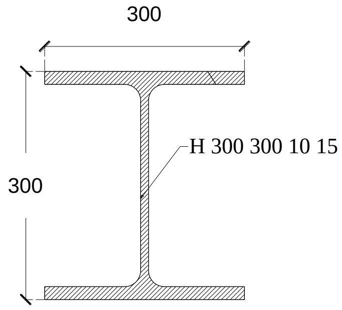
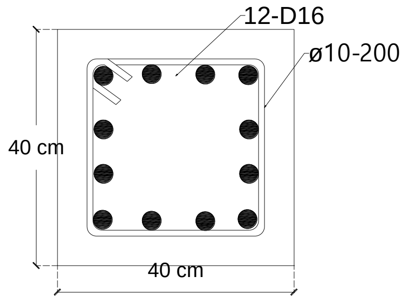
Table 9. Beam Comparison

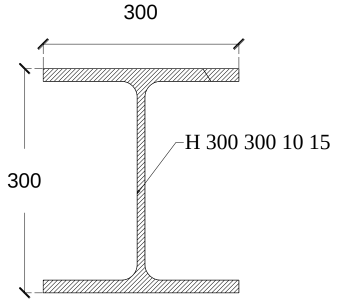
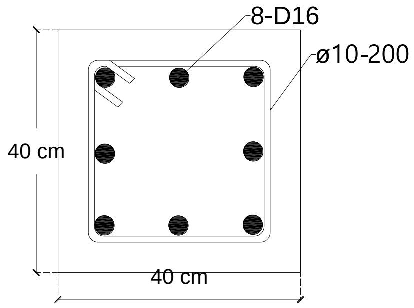
Beam	Steel Structure	Structure concrete	
		Focus	Field
Main			
	WF 300 150 5,5 8	B35/55	
Child			
	WF 250 125 6 9	B 25/35	

C. Column Comparison

For the comparison of beams on modified structures and existing structures dpata is seen as follows.

Table 10 Column comparison

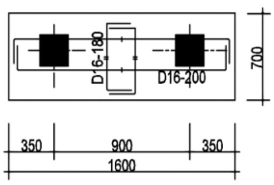
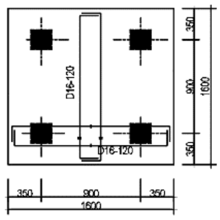
Column	Steel Structure	Concrete structure
1st Floor		

	H 300 300 10 15	B 40/40
2nd Floor		
	H 300 300 10 15	B 40/40

D. Foundation Comparison

Based on the recapitulation of the results of calculating the foundation of the modified structure (Steel structure) and the existing structure (Concrete structure) obtained with the same pole dimensions and pole depth the modified structure requires 2 piles, while for the existing structure it requires 4 poles to carry the working load. Details of the foundation comparison can be seen in Table 11.

Table 11 Foundation Comparison

-	Steel Structure	Concrete structure
Pole Details		
Mast Configuration	Pile (2 x 1)	Pile 2 x 2
Pole Dimensions	25 x 25	25 x 25
Depth	6 m	6 m
Thick Pile Cap	0,35 m	0,35 m
Pile Cap Dimensions	1.6 m x 0.7 m	1.6 m x 1.6 m
Flexor Reinforcement in the direction x	D16 - 180	D16 - 120
Y-direction bending reinforcement	D16 - 300	D16 - 120

4 CONCLUSIONS AND SUGGESTIONS

From the results of the calculations and analysis that have been carried out, the following conclusions can be drawn:

1.Upper structure planning includes secondary structures and primary structures, secondary structures include planning roof trusses, floor slabs, child beams, and stair structures, while primary structure planning includes main beam and column structures, where the strength and joints of each element are designed based on SNI 1729 2020, with the DFBK method

-Planning of the roof structure includes planning of the gording and roof easel where each easel is spliced with bolted joints, for gording using the profile C 100x50, whilen for the easel wears a profile 2L 45x45x4 , 2L 50x50x5 and 2L 30x30x3

-Office floor slabs use bondek from Super Floor Deck 0.75 mm thick with concrete slabs 110 mm thick, for negative reinforcement installed wire mesh M8 – 175.

-Planning of stairs with steel structures obtained the results of calculations used Stepping plates with a thickness of 4 mm with a support for profile plates L 50 x 50 x 6, for bordes used plates 8 mm thick with beam profile I 100 x 100 x 6 x 8, while the main beams of the stairs use a WF profile of 175 x 90 x 5 x 8.

-Planning of child beams and main beams with composite beams where the child beam uses a WF profile of 250×125×6×9, while in the main beam a WF profile of 300 x 150 x 5.5 x 8 is used in the longitudinal direction of the building. And the profile of WF 250×125 ×6× 9 in the transverse direction of the building.

-The main column planning is used profile H 300 x 300 x 10 x 15.

-Rigid Connection is used at the connection of the moment-retaining beams while Simple Connection is carried out at the connection of the beams. For the connection of the base plate, a plate with a thickness of 20 mm is used and 4 pieces of transport 16 diameter with a depth of 20 cm are installed.

2.In the planning of the lower structure including the planning of the pile foundation, pile cap, pedestal column, and sloof beam., the foundation carrying capacity planning is based on data from the *Cone Penetration Test* (CPT) test results used piles from PT WIKA with pile dimensions of 25 x 25 cm with a pile depth of 6 m with a total of 2 poles.

3.Comparing the results of the steel structure design against the existing concrete structure that has been recalculated shows that with the same structure configuration, the steel structure produces an effective weight of almost 50% lighter than the concrete structure so that the values of shear force, diplacement, and deviation between floors (*story drift*) in steel structures it is smaller compared to the initial structure in the form of reinforced concrete. In addition, because the weight of the resulting structure is lighter, the steel structure requires less pile foundation than the concrete structure, where the steel structure requires 2 poles while the concrete structure requires 4 poles.

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