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Design of the Abutments for the Kali Tlatar Pabelan Bridge

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Abstract

This research focused on the structural design of the abutments for the Kali Tlatar Pabelan Bridge, located on the Mendut–Tanjungpuan road section in Magelang Regency, Central Java. The bridge, with a total length of 40 meters and a width of 9 meters, was constructed to improve connectivity, support community mobility, and mitigate the impacts of cold lava flows from Mount Merapi. This study aimed to analyze and design the bridge's substructure, particularly the abutments, including their stability under working loads, reinforcement requirements, and foundation capacity. The research utilized primary data obtained from field surveys and secondary data derived from cone penetration test (CPT) results. Load planning and structural analysis were conducted based on Indonesian standards, including SNI 1725:2016, PPPJJR 1987, and SNI T-15-1991-03. Data processing was performed manually using Microsoft Excel, while structural drawings were prepared using AutoCAD. The analysis included the design evaluation of abutment walls, wing walls, foundations, and pile caps, along with reinforcement calculations for flexural and shear capacities. The results showed that the designed abutments met safety requirements for structural loads, stability, soil bearing capacity, eccentricity, and axial and lateral forces. The reinforcement designs for the abutment walls, wing walls, bored pile foundations, and pile caps were adequate to safely support the bridge structure. The application of structural analysis software, such as MIDAS, is recommended for improving efficiency in future bridge design planning. The bridge construction is expected to enhance regional connectivity, support economic activities, and contribute to disaster mitigation efforts in Magelang Regency and its surrounding areas.

INTRODUCTION

In general, bridges are structures that connect two separate locations separated by natural obstacles, such as rivers, straits, and ravines, as well as artificial obstacles, such as highways, railways, and drainage channels (Fletcher, 2024; Gonzalez et al., 2020; Labi, 2014; Singh et al., 2002). Bridge types based on function, location, construction materials, and structural systems have continued to develop rapidly in accordance with technological advancements, ranging from simple structures to advanced construction systems. This development aims to facilitate mobility and support economic activities in surrounding areas (Danladi et al., 2023; Junaid et al., 2025; Lättman & Otsuka, 2024; Papas et al., 2023; Szymańska & Nazarko, 2023).

The Tlatar Pabelan Bridge, located on the Mendut–Tanjungpuan Road section, connects the Yogyakarta and Magelang regions. The bridge project has a total length of 40.00 meters and a width of 9.00 meters, supported by two abutments. The road and bridge project is located in Magelang Regency, Central Java (Ahmad et al., 2026; Pangestu et al., 2026; Yudha et al., 2025).

The construction of the Tlatar Pabelan Bridge in Magelang Regency, Central Java, aims to improve connectivity and support community safety and economic activities. The bridge was constructed as an effort to mitigate the potential impacts of cold lava flows from Mount Merapi through the Progo River and Tlatar Pabelan River, while also facilitating vehicle movement between Yogyakarta and Magelang (Depari, 2022; Palmer, 2024; Thouret et al., 2023). In addition, the bridge is expected to reduce traffic congestion, particularly during holiday periods and Eid al-Fitr, as well as encourage economic growth in areas connected by the Mendut–Tanjungpuan Road, Muntilan, Magelang.

The abutment is a key component of the bridge substructure that functions to support loads from the superstructure, resist active earth pressure from the approach embankment (oprit), and transfer loads to the foundation. Therefore, its stability is essential to maintaining the safety and long-term performance of the bridge, particularly in areas vulnerable to natural hazards such as earthquakes, floods, and erosion (Buka-Vaivade et al., 2025; Han et al., 2025; Jami, 2025; Scala et al., 2025; Xi et al., 2026). Previous studies on abutment planning and analysis have been conducted by several researchers. Sulthon and Drajat (2025) analyzed the stability of the Meninting Bridge abutment in West Lombok and found that the safety factors against sliding (1.47) and overturning (1.37) did not meet the minimum required value of 1.5, while the maximum foundation stress (496.60 kN/m²) exceeded the allowable soil bearing capacity (359.55 kN/m²), indicating the need for substructure reinforcement. Sumardin et al. (2023) reviewed the design of the substructure of the Seri-Hukurila Bridge in Ambon by considering various loading conditions and concluded that two caisson foundations with a diameter of 3.0 meters provided adequate stability, with a safety factor of 1.5 against sliding and overturning. Siswanto et al. (2025) evaluated the stability of the Klagen Bridge abutment in Nganjuk using the Terzaghi method and Plaxis software, obtaining safety factors ranging from 2.00–2.20 and 3.82, which exceeded the minimum safety requirement of 1.5. Winangun et al. (2023) analyzed the external stability of the Penatih Bridge abutment and obtained safety factors for longitudinal sliding of 14.97, transverse sliding of 65.9, and soil bearing capacity of 2,716.22, which were significantly above the required limits. Research conducted on the Umpak Bridge in Malang Regency (2023) redesigned the reinforced concrete structure and obtained safety factors for sliding stability of 2.73 (>1.5), overturning stability of 4.33 (>2), and soil bearing capacity of 15.52 (>3), indicating that the abutment was safe based on all stability criteria.

Based on the literature review, a research gap was identified because most previous studies focused primarily on evaluating abutment stability under different soil conditions and loading characteristics. Meanwhile, studies specifically analyzing and redesigning the abutment of the Tlatar Pabelan River Bridge in Magelang by considering the potential impact of Mount Merapi cold lava flows remain limited. Furthermore, few studies have comprehensively integrated stability analysis, reinforcement calculations, and foundation design within a single research object using the latest SNI 1725:2016 standard. The novelty of this study lies in the redesign of the Tlatar Pabelan River Bridge abutment by considering cold lava flow-related loading conditions and local geotechnical characteristics based on cone penetration test (CPT) data. This study also applies an analytical approach combining manual calculations and column interaction diagrams to determine optimal reinforcement requirements, resulting in a safe, efficient, and standard-compliant abutment design according to applicable Indonesian regulations.

This research focused on the planning of the bridge substructure, particularly the abutment, including design analysis, structural stability evaluation, and foundation calculations. The purpose of this study was to reevaluate the reinforced concrete abutment design to ensure compliance with strength, safety, and efficiency requirements based on applicable Indonesian standards. In addition to producing an appropriate structural design, this research is expected to contribute additional knowledge and practical experience in bridge abutment planning as part of transportation infrastructure development.

METHOD

General Description

This chapter describes the research method used for the planning of the Tlatar Pabelan River Bridge abutment on the Mendut–Tanjungpuan Road section, Magelang, Central Java. The research method included data collection, data processing, and analysis stages, which were presented through a flowchart. This study aimed to analyze abutment stability, determine an appropriate foundation type, and evaluate the structural capacity of the abutment under working loads.

Data Source

The data used in the study consisted of primary data and secondary data. Primary data was obtained through field surveys and interviews to find out the existing condition of the bridge. Meanwhile, secondary data was obtained from related agencies, mainly in the form of soil data from the cone penetration test (CPT). The data is used to determine the bearing capacity of the soil, determine the depth of the foundation, and choose the type of foundation that suits the field conditions.

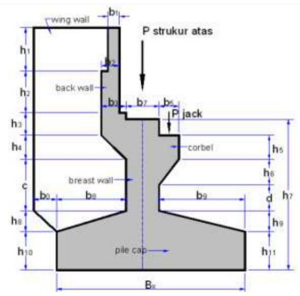
Data Processing Methods

Data processing is carried out based on relevant theories and regulations, then analyzed using manual calculations with the help of Microsoft Excel and drawing preparation using AutoCAD. The data from the field survey is used as a basis for analysis and planning of the abutment. Load planning refers to SNI 1725:2016, PPPJJR 1987, and Decree SNI T-15-1991-03 as guidelines in calculating loads and planning abutment structures. All stages of research, from data collection to analysis and planning, are presented in a research flowchart.

RESULTS AND DISCUSSION

Abutment Repeat Analysis

Calculation of Longitudinal Rebar Abutment Wall Reinforcement



Concrete grade(K) = 300
Mutu Low (U) = 39
Compressive strength of concrete (f_c') = 24.9 MPa
Steel melting stress (f_y') = 390 Mpa
Total height of the abutment (L) = 6.5 m
Abutment wall dimensions (h) = 1.5 m
By = 10 m
Reviewed 1 meter wide abutmnet wall
Abutment wall width (b) = 1,000 mm
Abutment wall thickness (h) = 1,500 mm
Cross-sectional area of the abutment wall under review (A_g):

Ag = b x h

= 1,000 mm x 1,500 mm

= 1.500.000 mm2

Pu = ultimate axial force on the pier column (kN)

Mu = the ultimate moment on the pier column (kN)

Pu = P.Pn

Mu = F.Mn

$\alpha = \Phi.Pn / (fc' \times A_g) = Pu \times 103 / (fc' \times Ag)$

$\beta = \Phi.Mn / (fc' \times Ag \times h) = Mu \times 106 / (fc' \times A_{g\ x}\ D)$

Table 1. Load Combinations

No	Load Combinations	Load Analysis Results		For 1 m Width		a	b
		Pu (kN)	Mu (kN)	Pu (kN)	Mu (kN)		
1	Kombinasi -1	28.463,02	13.811,70	2.846,30	1.381,17	0,0785	0,02613
2	Kombinasi -2	28.733,42	16.936,70	2.873,34	1.693,67	0,0792	0,03204
3	Kombinasi -3	28.463,02	28.422,41	2.846,30	2.842,24	0,0785	0,05378
4	Kombinasi -4	28.783,82	28.650,78	2.878,38	2.865,08	0,0794	0,05421
5	Kombinasi -5	26.552.62	49.249,30	2.655,26	4.924,93	0,0732	0,09318

Source: Author's calculation based on SNI 1725:2016, 2024.

The distance of reinforcement to the outer side of the concrete,

d’ = 50 mm

h' = h-2 x d' = 1.400 mm

h' / h = 0,8

The values $\alpha = \Phi.Pn / (fc' \times Ag)$ and $\beta = \Phi.Mn / (fc' \times Ag \times h)$ are plotted into the diagram

From the column interaction diagram is obtained:

Required reinforcement ratio r = 0,5%

Required reinforcement area $As – \rho \times A_g$ = 7.500 mm2

Diameter of the reinforcement used D = 32 mm

d = h – z = 1.400 mm

Press reinforcement is made the same as tensile reinforcement,

$As_{(tekan)} = As_{(tarik)} = 1/2 \times A_s$ = 3.700 mm2

Required reinforcement distance,

$s = 1/2 \times \pi/4 \times D2 \times b/(1/2 \times A_s)$ = 214,47 mm

Longitudinal reinforcement used(s) = 200 mm

Table 2. Longitudinal reinforcement

Reinforcement Type	Number of Layers	Reinforcement Diameter (D) (mm)		Distance(s) (mm)	ρ = As/bxd	
Longitudinal Reinforcement Press	1	D	32	200	ρ press =	0,27%
Tulangan Longitudinal Tarik	1	D	32	200	ρ tarik =	0,27%
					ρ total =	0,54%

Source: Author's calculation based on structural analysis, 2024.

(ok)

Total longitudinal reinforcement area used,

$As_{tekan} = \pi/4 \times D2 \times (b/s)$ = 3.021,24 mm2

$As_{tarik} = \pi/4 \times D2 \times (b/s)$ = 4.021,24 mm2

$As = As_{Press + Pull\ Axle}$ = 8.042, 48 mm2

For the reinforcement of the press is taken 20% of the longitudinal reinforcement of the press,

As_{tekan}'= 20% x $A_{S\ Press}$' = 804,25 mm2
The diameter of the reinforcement used, D = 16 mm2
Required reinforcement distance, s = (π/4 x D2xb) / Ace' = 250 mm2
Used reinforcement, D16-200 1 lapis

Spacing between reinforcements is qualified

For reinforcement for tensile is taken 20% longitudinal tensile reinforcement,
As_{tarik}'= 20% x $A_{s\ tarik}$' = 804,25 mm^2
The diameter of the reinforcement used, D = 16 mm2
Required reinforcement distance, s = (π/4 x D2xb) / Ace' = 250 mm2
Used reinforcement, D16-200 1 lapis

Spacing between reinforcements is qualified

For longitudinal press reinforcements are used,
As' = 30% x A = 2412,74 mm2
Used reinforcement, D = 19 mm2
n = 8.5
9 D 19

Calculation of Abutment Wall Shear Reinforcement

The calculation of shear reinforcement for an abutment wall is based on the ultimate axial moment and force for the decisive load combination in the calculation.

Axial force ultimates plan Pu = 2,878.4 kN = Pu = 2,878,381,58 N
Ultimate style plan Mu = 2,865.1 kN.m = Mu = 2,865,078.30 N.mm
Concrete Quality K = 300 = fc' = 24,9 Mpa
Steel Quality U = 39 = f_y = 390 Mpa
Reviewed wide abutment walls b = 1.000 mm
Shear force reduction factor F = 0.6
Height of the abutment wall L = 6,500 mm
Thick wall abutment h = 1,500 mm
Longitudinal reinforcement area of abutment wall As = 8.042,48 mm2
Reinforcement distance to the outer side of concrete d = 50 mm
Ultimate moment-consequence shear force Vu = Mu/L = 440.78 N
D = h – d = 1,450 mm
Vcmax = 0.2 x fc' x b x d = 7.221 N
4,332.60 > seen **(Ok)**
b1 = 1.4 – d / 200 = 0.675 < **1 then taken**
β2 = 1 + Pu / (14 x fc' x b x h) = 1 B1 = 0,69
b3 = 1
Vuc = b1 x b2 x b3 x b x d [A_s x fc'/(bxd)]$^{1/3}$ = 450.738.14 N
Vc = Vuc + 0,6 x b x d = 480.738.14 N
Vc = 0,3 x (√fc') x b x d x √[1+0,3 x Pu/(bxd) = 945.459.74 N
Taken, Vc = 945,459.74 N mass, Φ x Vc = 567.275.85 N

Φ x Vc > Vu (only minimum shear reinforcement needed)

The shear force is 50% borne by the shear reinforcement, Vs = 50% x (Vu/ Φ) = 367.32 N
For shear reinforcement is used diameter, D = 16 mm
The shear reinforcement distance in the direction of the Y-axis,
and = 400 mm
Shear reinforcement area, Asv = π/4 x D2 x b/sy = 502,65 mm2
The required shear reinforcement distance (x-axis direction),
sx = Asv x fy x d/Vs = 773.856,75 mm
Used shear reinforcement, D16 - 400 / 400

The distance between reinforcements is OK

Bottom Back Wall Repeating Analysis

Courage concrete, K = 300

Low quality, U = 39

Compressive strength of concrete fc' = 24.9 Mpa

Steel melting stress fy' = 390 Mpa

Dimensions; Thick h = 1.45 m

Width By = 10 m

Moment of treatment Mu = 162.66 kN.m

Vu = 55.40 kN.m

Reviewed in broad I m, then Mu = 16.27 kN.m

Vu = 5.54 kN.m

a) Calculation of Lower Back Wall Bending Reinforcement

The ultimate plan moment Mu = 16.265625 kN.m

Compressive strength of concrete fc' = 24,9 Mpa

Steel Melting Tension fy' = 390 Mpa

Thick concrete h = 600 mm

Reinforcement distance to the outer side of concrete d' = 50 mm

Steel modulus of elasticity Es = 200,000

Form factors of concrete stress distribution b1 = 0.85

$r_b = \beta 1 \times 0.85 \times f_c' / f_y \times 600 / (600+fy) = 0{,}0280$

$Rmax = 0{,}75 \times \rho_b \times f_y \times [1-0.5x0.75x\ \rho_b \times f_y/(0.85 \times f_c')] = 6{,}598$

Reducing Strength Factor φ = 0.8

Shear force reduction factor F = 0.6

Effective thickness d = h – d = 550 mm

Width reviewed b = 1.000 mm

Nominal moment of the plan Mn = Mu / φ = 20.332 kN.m

Moment holding factor Rn = Mn x 106 / (bxd2) = 0,067

Required reinforcement ratio

$\rho = 0.85 \times fc'/fy \times [1-\sqrt{1} - 2 \times Rn / (0.85 \times fc')] = 0.00017$

Minimum reinforcement ratio, ρmin =25% x 1,4/fy = 0,00090

The ratio of reinforcement used, ρ = 0.00017262

The required area of reinforcement, As = ρ x w x d = 94.94 mm2

Diameter of the reinforcement used D = 19 mm

Reinforced distance used, $s = (\pi/4 \times D^2 \times w) / As = 2986.43$ mm

Used reinforcement, D 19 -200

Spacing between reinforcements is qualified

As = (π/4 x D2 x b)/s = 1.417,64 mm2

For reinforcement to be taken 50% of the tree reinforcement,

As' = 50% x Ace = 47,470 mm2

The diameter of the reinforcement used, D = 13 mm

Required reinforcement distance, s = (π/4 x D2 x b) / As = 2796.16 mm

Used reinforcement, D 13 - 200

As' = (π/4 x D2 x b)/s = 663,661 mm2

b) Calculation of Lower Back Wall Sliding Reinforcement

Gaya geser ultimit, Vu = 5.54 N

Shear force reduction factor, F = 0.6

Ultimate shearing capacity Vcmax = 5/6 x φ x(√fc') x b x d = 1,372.45 N

Vcmax > V_u (OK), Dimension safe against shear

The shear force held by concrete, $V_c = 1/6 \times (fc'^{1/2}) \times w \times d$ = 457.415, 75 N

$F \times V_c$ = 274.449.45 N

F x V_c > Vu (only need minimum shear reinforcement)

The shear force held by the shear reinforcement, V_s = 50% x Vu = 2.77 N
For shear reinforcement is used diameter, D = 13 mm
The distance of the shear reinforcement in the direction of the y-axis,
and = 400 mm
Shear reinforcement area, A_{sv} = w/4 x D2 x w/sy = 331.83 mm^2
The required shear reinforcement distance (x-axis direction),
sx = A_{sv} x f_v x d/vs = 25,695,627.19 mm
Used shear reinforcement, D 13 – 400 / 400

The distance between reinforcements is OK

Upper Back Wall Repetition Analysis

Concrete quality, K = 300
Low quality, U = 39
Compressive strength of concrete fc' = 24.9 Mpa
Steel melting stress fy' = 390 Mpa
Dimensions; Thick h = 0.3 m
Width By = 10 m
Moment of treatment Mu = 45.92 kN.m
Vu = 230.74 kN
Reviewed in broad I m, then Mu = 45.9 kN.m
Vu = 23.07 kN

a) Calculation of Upper Back Wall Bending Reinforcement

The ultimate plan moment Mu = 4.59 kN.m
Compressive strength of concrete fc' = 24.9 Mpa
Steel Melting Tension fy' = 390 Mpa
Thick concrete h = 300 mm
Reinforcement distance to the outer side of concrete d' = 50 mm
Steel modulus of elasticity Es = 200,000
Form factors of concrete stress distribution b1 = 0.85

$\rho_b = \beta 1 \times 0.85 \times fc'/ f_y \times 600 /(600+f_y) = 0.0280$

$Rmax = 0.75 \times \rho_b \times f_y \times [1-0.5x0.75x \rho_b \times f_y/(0.85 \times f_c')] = 6.598$

Reducing factor of bending strength, φ = 0.8
Shear force reduction factor, F = 0.6
Effective thickness, d = h – d' = 250 mm
Width reviewed, b = 1,000 mm
The nominal moment of the plan, Mn = Mu / φ = 5.7396 kN.m
Moment holding factor, Rn = Mn x 106 / (bxd2) = 0,0918

Rn < Rmax (OK)

Required reinforcement ratio,
ρ = 0.85 x fc'/fy x [1-√1 – 2 x Rn / (0.85 x fc')] = 0.000236
Minimum reinforcement ratio, ρmin =25% x 1.4/fy = 0.0009
The ratio of reinforcement used, ρ = 0.0009
The required area of reinforcement, As = ρ x b x d = 224,359 mm2
Diameter of the reinforcement used D = 16 mm
Reinforced distance used, s = (π/4 x D2 x w) / As = 896.16 mm
Used reinforcement, D 16 -200

Spacing between reinforcements is qualified

As = π/4 x D2 x b/s = 1005.310 mm2
For reinforcement to be taken 50% of the tree reinforcement,
As' = 50% x Ace = 502.548246 mm2

The diameter of the reinforcement used, D = 13 mm
Required reinforcement distance, s = (π/4 x D2 x b) / As = 264.0625 mm
Used reinforcement, D 13 – 200

Spacing between reinforcements is qualified

As' = π/4 x D2 x b/s = 663,661 mm2

b) Calculation of Lower Back Wall Sliding Reinforcement

Gaya geser ultimit, Vu = 23.0735 N
Shear force reduction factor, F = 0.6
Ultimate shear capacity Vcmax = 5/6 x φ x(√fc') x w x d = 623,748.75 N

Vcmax > Vu (OK), Dimension safe against shear

The shear force held by concrete, Vc = 1/6 x (fc'1/2) x w x d = 207.916.25 N
F x Vc = 124.749.75 N

Φ x Vc > Vu (only minimum shear reinforcement needed)

The shear force held by the shear reinforcement, Vs = 50% x V_u = 11.53675 N
For shear reinforcement is used diameter, D = 13 mm
The distance of the shear reinforcement in the direction of the y-axis,
and = 400 mm
Shear reinforcement area, A_{sv} = w/4 x D2 x w/sy = 331.830724 mm^2
The required shear reinforcement distance (x-axis direction),
sx = A_{sv} x f_y x d/vs = 2,804,385.60 mm
Used shear reinforcement, D 13 – 400

The distance between reinforcements is OK

Wing Wall Repetition Analysis

a) Vertical Direction Wing Wall Overview

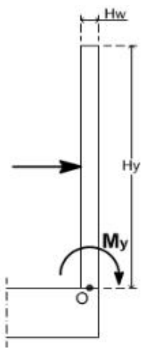


Figure 1. Moment on the clamp
Source: Author's analysis and design drawing, 2024.

Concrete quality, K = 300
Low quality, U = 39
Compressive strength of concrete, fc' = 24.9 Mpa
Steel melting stress, fy' = 390 Mpa
Dimensions: Thickness Hw = 0.55 m
Hy width = 1.8 m
Ultimite Mu = 30.6504 kN.m
Vu = 42.57 kN
Viewed as a width of I m, then Mu = 17.03 kN.m
Vu = 23.65 kN

b) Calculation of Wing Wall Bending Reinforcement

The ultimate plan moment $Mu = 17.03$ kN.m

Compressive strength of concrete $fc' = 24.9$ Mpa

Steel Melting Tension $fy' = 390$ Mpa

Thick concrete $h = 500$ mm

Reinforcement distance to the outer side of concrete $d' = 50$ mm

Steel modulus of elasticity $Es = 200,000$

Form factors of concrete stress distribution $b1 = 0.85$

$$\rho_b = \beta 1 \times 0.85 \times fc'/ f_y \times 600 /(600+f_y) = 0.0280$$

$$Rmax = 0.75 \times \rho_b \times f_y \times [1-0.5 \times 0.75 \times \rho_b \times f_y/(0.85 \times f_c')] = 6.598$$

Reducing factor of bending strength, $\varphi = 0.8$

Shear force reduction factor, $F = 0.7$

Effective thickness, $d = h - d' = 450$ mm

Width reviewed, $b = 1,000$ mm

The nominal moment of the plan, $Mn = Mu / \varphi = 21.29$ kN.m

Moment holding factor, $Rn = Mn \times 106 / (bxd2) = 0,105$

Rn < Rmax (OK)

Required reinforcement ratio,

$$\rho = 0.85 \times fc'/fy \times [1-\sqrt{1} - 2 \times Rn / (0.85 \times fc')] = 0.00027$$

Minimum reinforcement ratio, $\rho min = 25\% \times 1.4/f_y = 0.0009$

The ratio of reinforcement used, $\rho = 0.0009$

The required area of reinforcement, $As = \rho \times w \times d = 403.85$ mm2

Diameter of the reinforcement used $D = 25$ mm

Reinforcing distance used, $s = (\pi/4 \times D^2 \times w) / As = 896.16$ mm

Used reinforcement, D 25 -200

Spacing between reinforcements is qualified

$As = \pi/4 \times D2 \times b / s = 2,454.37$ mm2

For reinforcement to be taken 50% of the tree reinforcement,

$As' = 50\% \times Ace = 1,227.18$ mm2

The diameter of the reinforcement used, $D = 19$ mm

Required reinforcement distance, $s = (\pi/4 \times D2 \times b) / As = 231.04$ mm

Used reinforcement, D 19 – 200

Spacing between reinforcements is qualified

$As' = \pi/4 \times D2 \times w/s = 1,417.64$ mm2

c) Calculation of Wing Wall Sliding Reinforcement

Gaya geser ultimit, $Vu = 23,650$ N

Shear force reduction factor, $F = 0.6$

Ultimate shear capacity $V_{cmax} = 5/6 \times \varphi \times (\sqrt{fc'}) \times b \times d = 1.122.747.75$ N

Vcmax > V_u (OK), Dimension safe against shear

The shear force held by concrete, $V_c = 1/6 \times (fc'^{1/2}) \times b \times d = 1,481,131.32$ N

$F \times V_c = 204,984.76$ N

F x V_c > Vu (only need minimum shear reinforcement)

The shear force held by the shear reinforcement, $Vs = 50\% \times V_u = 11,825$ N

For shear reinforcement is used diameter, $D = 13$ mm

The distance of the shear reinforcement in the direction of the y-axis,

and = 400 mm

Shear reinforcement area, $A_{sv} = w/4 \times D2 \times w/sy = 331.830724$ mm^2

The required shear reinforcement distance (x-axis direction),

$sx = A_{sv} \times f_y \times d/vs = 2,462.422498$ mm

Used shear reinforcement, D 13 – 400

The distance between reinforcements is OK

d) Horizontal Direction Wing Wall Overview

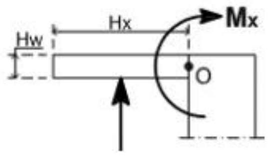


Figure 2. Horizontal Direction Moment
Source: Author's analysis and design drawing, 2024

Concrete quality, K = 300
Low quality, U = 39
Compressive strength of concrete, fc' = 24.9 Mpa
Steel melting stress, fy' = 390 Mpa
Dimensions: Thickness Hw = 0.5 m
Hy width = 2.75 m
Moment of treatment Mu = 46,827 kN.m
V_u = 42.57 kN
Viewed as 1 m wide, M_u = 17,028 kN.m
Vu = 15.48 kN

e) Calculation of Wing Wall Bending Reinforcement

The ultimate plan moment Mu = 17,028 kN.m
Compressive strength of concrete fc' = 24.9 Mpa
Steel Melting Tension fy' = 390 Mpa
Thick concrete h = 500 mm
Reinforcement distance to the outer side of concrete d' = 50 mm
Steel modulus of elasticity Es = 200,000
Form factors of concrete stress distribution b1 = 0.85

$r_b = \beta 1 \times 0.85 \times fc'/ f_y \times 600 /(600+f_y) = 0.0280$

$Rmax = 0.75 \times \rho_b \times f_y \times [1-0.5 \times 0.75 \times \rho_b \times f_y/(0.85 \times f_c')] = 6.598$

Reducing factor of bending strength, φ = 0.8
Shear force reduction factor, F = 0.6
Effective thickness, d = h – d' = 450 mm
Width reviewed, b = 1,000 mm
The nominal moment of the plan, Mn = Mu / φ = 21,285 kN.m
Moment holding factor, Rn = Mn x 106 / (bxd2) = 0,105

Rn < Rmax (OK)

Required reinforcement ratio,

$\rho = 0.85 \times fc'/fy \times [1-\sqrt{1} - 2 \times Rn / (0.85 \times fc')] = 0.00027$

Minimum reinforcement ratio, $\rho min = 25\% \times 1.4/f_y = 0.0009$
The ratio of reinforcement used, ρ = 0.0009
The required area of reinforcement, As = ρ x w x d = 403.85 mm2
Diameter of the reinforcement used D = 25 mm
Reinforced distance used, $s = (\pi/4 \times D^2 \times w) / As = 1{,}215.50$ mm
Used reinforcement, D 25 -200

Spacing between reinforcements is qualified

As = π/4 x D2 x b / s = 2,454.37 mm2

For reinforcement to be taken 50% of the tree reinforcement,

As' = 50% x Ace = 1,227.18 mm2

The diameter of the reinforcement used, D = 18 mm

Required reinforcement distance, s = (π/4 x D2 x b) / As = 207.36 mm

Used reinforcement, D 16 – 200

Spacing between reinforcements is qualified

As' = π/4 x D2 x b/s = 1,272.35 mm2

f) Calculation of Wing Wall Sliding Reinforcement

Gaya geser ultimit, Vu = 15.480 N

Shear force reduction factor, F = 0.6

Ultimate shear capacity V_{cmax} = 5/6 x φ x(√fc') x b x d = 112.274.77 N

Vcmax > V_u (OK), Dimension safe against shear

The shear force held by the concrete, V_c = 1/6 x ($fc'^{1/2}$) x w x d = 374.249.25 N

F x V_c = 224.549.55 N

F x V_c > Vu (only need minimum shear reinforcement)

The shear force held by the shear reinforcement, Vs = 50% x V_u = 7.740 N

For shear reinforcement is used diameter, D = 13 mm

The distance of the shear reinforcement in the direction of the y-axis,

and = 400 mm

Shear reinforcement area, A_{sv} = w/4 x D2 x w/sy = 331.83 mm^2

The required shear reinforcement distance (x-axis direction),

sx = A_{sv} x f_y x d/Vs = 3.762 mm

Used shear reinforcement, D 13 – 400 /400

The distance between reinforcements is OK

Calculation Result Picture
Abutment Wall Repeats

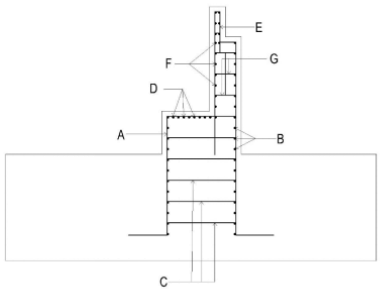


Figure 3. Abutment reinforcement
Source: Author's design based on structural analysis, 2024

Abutment recurrence data:

A =	D	32	-	200		
B =	D	16	-	200		
C =	D	16	-	400	/	400
D = 9 D 19						
E =	D	19	-	200		
F =	D	13	-	200		
G =	D	13	-	400	/	400

Wing Wall Abutment Repeats

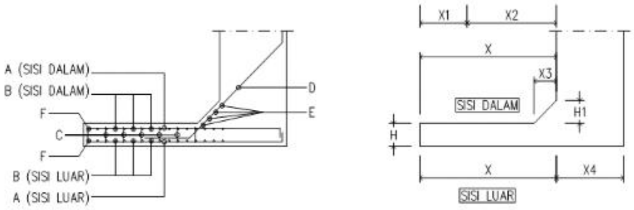


Figure 4. Reinforcement Wing Wall
Source: Author's design based on structural analysis, 2024.

Wing wall repeating data:

A (Inner Side) =	D	25	-	200		
A (Outer Side) =	D	19	-	200		
B (Inner Side) =	D	25	-	200		
B (Outer Side) =	D	19	-	200		
C =	D	13	-	400	/	400
D =	D	25	-	200		
E =	D	25	-	200		
F =	D	19	-	200		

Foundation Reinforcement Analysis

Foundation Data

a) Foundation Materials/Materials

Table 3. Foundation Materials and Materials

Compressive strength of concrete	**K-350 = fc'**	**29,05 Mpa**
Melting stress	fy'	390 Mpa
Modulus Elasticity of concrete	Ec = 4.700 x $\sqrt{f_c}$'	25.332,08 Mpa
Specific gravity of reinforced concrete	γ	25 kN/m3
Soil volume weight	γ	17.2 kN/m3
Angle of friction in the ground	φ	35th
Soil Cohesion	C	0.36 kN/m2

Source: Author's calculation based on SNI 1725:2016 and soil data, 2024

b) Palm Pile Cap Dimensions

Table 4. Dimensions of Pilecap Palm B_x

Lebar arah x (bottom)	**Bx**	**9,2 m**
Lebar arah y (bottom)	By	9,2 m
Thick pile cap end	Hp	1,25 m
Thick pile cap base	Ht	1,25 m
Diameter Fondasi	D	2,5 m
Depth of Foundation	L	2 m
Number of x-axis directional foundations	nx	3 pieces
Number of Y-axis directional foundations	The	2 pieces
Distance Foundation direction x	x	4 m
Distance Foundation direction y	y	8 m
Distance of the Foundation to the edge of the pile stamp direction x	a	0,6 m
Foundation Distance to Y-Direction Cap Pile Edge	b	0,6 m

Source: Author's design based on structural analysis, 2024

Foundation Ironing

Table 5. Recapitulation of Foundation Analysis Results

1	**Axial bearing capacity of the foundation**	**Pijin**	**19,268.96 kN**
2	Lateral bearing capacity of the foundation	Hijin	628.15 kN
3	Maximum moment Foundation	Mmax	68,080.47 kN.m

Source: Author's calculation based on bearing capacity and structural analysis, 2024

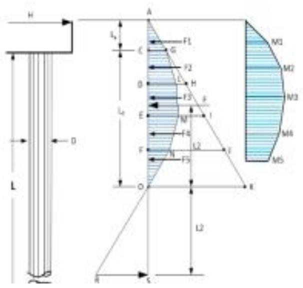
a) Longitudinal Reinforcement Bending Press
Maximum axial force on the foundation, Pmax = P_{ijin} = 19,268.96 kN
Maximum moment on the foundation, Mmax = 68.080,47 kN.m
Baby factor of heating, K = 1,2
Concrete quality K-350, fc' = 29.05 Mpa
Melting Voltage U-39, f_y' = 390 Mpa
Ultimate axial force, F x P_n = Pu = K x Pmax = 23,122.75 kN
Moment of heating, F x M $_n$ = Mu = K x M_{max} = 81,696.56 kN.m
Diameter founded, D = 2,500 mm
The cross-sectional area of the foundation, Ag = (π/4 x D2) = 4,908,738.52 mm2

α = Φ x P_n / (fc' x Ag) = Pu x 103 / (fc' x Ag) = 0,162152439
β = Φ x Mn / (fc' x Ag x D) = Mu x 106 / (fc' x Ag x D) = 0,229164708
d = 122.5 mm
The distance of reinforcement to the outer side of the concrete, d = D – 2 x d' = 2.255 mm
d/D = 0.902
The values α = Φ x P_n / (fc' x Ag) and β = Φ x Mn / (fc' x Ag x D) are plotted into the column interaction diagram as follows:
From the column interaction diagram obtained,
Required reinforcement ratio, ρ = 1.00%
Area of reinforcement used A_s = ρ x Ag = 49.087,39 mm2
The diameter of the reinforcement used, D = 32 mm
The amount of reinforcement required, n = If / (π.4 x D2) = 61.035156 buah
Used reinforcement, 70 D 32
Installed reinforcement area, As = n x π.4 x D^2 = 56.297,34 mm2
Installed reinforcement ratio, ρ = As / A_g = 114.69% **(OK)**

b) Foundation Shear Reinforcement
The calculation of shear reinforcement The foundation is based on axial momentum and force for the decisive combination of loads in the calculation of axial reinforcement, compressive, compressive and flexural.
Foundation length L = 2,000 mm
Diameter Merged D = 2,500 mm
The area of foundation reinforcement, Axle = 56,297.34 mm2
Compressive strength of concrete, f_c' = 29.05 Mpa
Steel melting stress, f_y = 390 Mpa
Baby factor of heating, K = 1,2
Nominal axial force, Pyin = 19,268.96 kN
Momen nominal, Mijin = 68.080,47 kN.m
Gaya lateral nominal, Hijin = 628,1530921 kN
Ultimate axial force, Pu = 23.122.75 kN = 23.122.752 N
At the moment of examination, Mu = 81,696.56 kN.m = 81,696,561.76 N.mm

Gaya lateral ultimit, H_{ijin} = 628.1530921 kN = 628.153.09 N
Shear force reduction factor, F = 0.6
Ultimate shear force due to shear, Vu = Mu / L = 40.848.28 N
Shear force due to lateral force, $V_u = K \times H_{ijin}$ = 753.783.71 N
Taken the ultimate shear force of the plan, Seen = 753.783.71 N
The distance of reinforcement to the outer side of the concrete, d = 122.5 mm
d = D – d = 2,377.50 mm
V_{cmax} = 0.3 x fc' x b x d = 41.439.825 N
F x V_{cmax} = 24,863,895 N > Seen **(OK)**
β1 = 1.4 – d / 2000 = 0.21125 < 1 then take 0.21125
β2 = 1 + Pu / (14 x fc' x w x h) = 1.011370913
b3 = 1
Vuc = β1 x β2 x$_{\beta 3}$ x D x d x [As x Fc' / (D x d)]$^{1/2}$ = 666,123.03 N
Vc = V_{uc} + 0.6 x D x d = 4.232.373.03 N
F x V_c = 2,539,423.82 N
Φ x V_c > **Vu (only minimum shear reinforcement required)**
The sliding force carried by the sliding tunagn,
Vs = [(Vu – Φ x Vc)/Φ] = 1,786,640.11 N
For sliding reinforcement is used a spiral shank with a diameter, 2 D 10
Shear reinforcement area, Asv = π/4 x D^2 x n = 157.080 mm2
Required shear reinforcement distance, $S_x = A_{sv} \times f_y \times d/Vs$ = 81,566 mm
Used sliding reinforcement, 2 D 10 – 75
Spacing between reinforcements is qualified

Pembesian Pile Cap



a) Foundation Ultimate Axial Force
Number of foundations : n = 6 pieces
Total direction x : nx = 3 pieces
Number of directions y : ny = 2 pieces
Directional distance x = 4 m
Directional distance y = 8 m

Table 6. Ultimate Axial Force Foundation y2^2

No	Xmax	4	m		Ymax	8	m	
1	x1	8	x12	64	y12	8	y12	64
2	x2	4	x22	16	y22	0	y22	0
3	x3	0	x32	0	y32	0	y32	0
			∑ X2	80			∑ Y2	64

Source: Author's calculation based on load combinations, 2024

Load combination recap for ultimate load planning

Table 7. Ultimate Load Combination Recap

No	Load Combinations	Excessive Voltage	P (kN)	Tx (kN)	You (kN)	Mx (kNm)	My (kNm)
1	Kombinasi – 1	0%	28.463,02	10.120,85	102,97	22.883,92	1.232,28
2	Kombinasi – 2	25%	28.733,42	11.252,45	0	34.401,71	0
3	Kombinasi – 3	40%	28.463,02	11.225,45	102,97	34.095,58	1.232,28
4	Kombinasi – 4	40%	28.783,82	10.120,85	102,97	22.916,00	1.232,28
5	Kombinasi – 5	50%	26.552,62	18.310,71	4.453,84	85.100,59	33.401,62

Source: Author's calculation based on SNI 1725:2016, 2024

Load Overview in the X Direction
The maximum and minimum axial forces carried by a single Foundation:
Pumax = P_u / n + Mux * Xmax / ΣX^2
Pumin = P_u / n - Mux * Xmax / ΣX^2
Maximum and Minimum Axial Force Tabulation Table

Table 8. Combination of Maximum and Minimum Axial Force X Direction

No	Kombinasi Charging	Pu (kN)	Mux (kN.m)	Pu/n (kN)	Mux * X / ΣX2 (kN)	Pumax (kN)	Pumin (kN)
1	Kombinasi – 1	28.463,02	22.883,92	4.743,84	1.144,20	5.888,03	3.599,64
2	Kombinasi – 2	28.733,42	34.401,71	4.788,90	1.720,09	6.508,99	3.068,82
3	Kombinasi – 3	28.463,02	34.095,58	4.743,84	1.704.78	6.448,61	3.039,06
4	Kombinasi – 4	28.783,82	22.916,00	4.797,30	1.145,80	5.943,10	3.651,50
5	Kombinasi – 5	26.552,62	85.100,59	4.425,44	4.255,03	8.680,47	170,41

Source: Author's calculation based on structural analysis, 2024

Load Overview in the Y Direction
Maximum and minimum axial forces carried by one Foundation :
Pumax = Pu / n + Muy * Ymax / ΣY2
Pumin = Pu / n - Muy * Ymax / ΣY2

Table 9. Combination of Maximum and Minimum Axial Force of Y Direction

No	Kombinasi Charging	Pu (kN)	Very (kN.m)	Pu/n (kN)	Very *X/ΣY2 (kN)	Pumax (kN)	Pumin (kN)
1	Kombinasi – 1	28.463,02	1232,280788	4.743,84	154,035	4.897,87	4.589,80
2	Kombinasi – 2	28.733,42	0	4.788,90	0	4.788,90	4.788,90
3	Kombinasi – 3	28.463,02	1232,280788	4.743,84	154,035	4.897,87	4.589,80
4	Kombinasi – 4	28.783,82	1232,280788	4.797,30	154,035	4.951,34	4.643,27
5	Kombinasi – 5	26.552,62	33401,61881	4.425,44	4175,202	8.600,64	250,23

Source: Author's calculation based on structural analysis, 2024

Foundation's Ultimate Shear Moment and Force

Notasi	(m)	Notasi	(m)
H1	1,8	b1	0,45
H2	0,3	b2	0,65
H3	1,45	b3	0,85
H4	0,7		
H5	0,8	b5	0
H6	0,7		
H7	10,15	b7	1,5
H8	0,5	b8	3,6

Notasi	(m)	Notasi	(m)
H9	0,5	b9	3,4
h10	1,25	b10	0,5
H11	1,25		
c	6,5	Bx	8,5
d	6,5		
Remarks			
Abutment Length		By	10
Tebal Wing-wall		hw	0,55
Depth of Foundation		Df	1,75

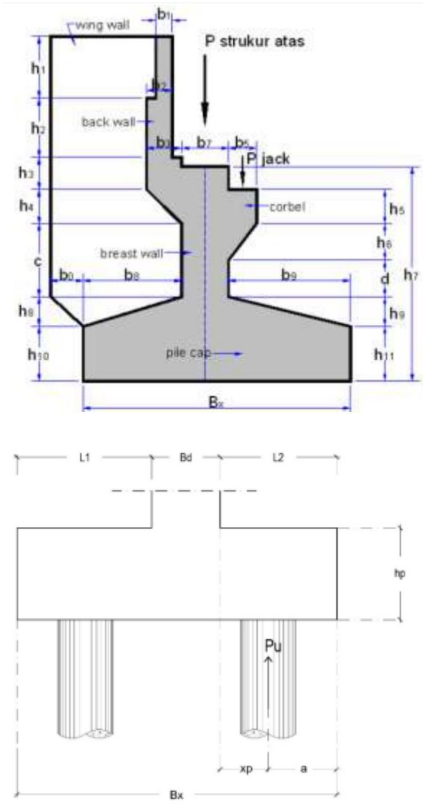


Figure 5. Foundation Ultimate Shear Force
Source: Author's analysis and design drawing, 2024

Pile cap dimension data:

Bx = 9,2 m
Bd = 1,5 m
L1 = 3,6 m
L2 = 3,4 m
xp = 1,55 m
a = 1,85 m
hp = 1,75 m

Maximum ultimate bearing force (plan) on Foundation, P_{umax} = 8,600.64 kN
Concrete quality, K-350 fc' = 29,05 Mpa
Steel melting stress, U – 39 f_y' = 390 Mpa
Specific gravity of reinforced concrete, c_c = 25 kN/m^3

Specific gravity of the heapbed, c_W = 17.2 kN/m^3

Table 10. Earthquake load distribution on the abutment

No	Weight Wt (kN)	TEQ (kN)	Description of the arm against the point O	Distance and (m)	MEQ (kNm)
ABUTMENT					
8	225	49,61	y8 = h10 + 1/3 x h8	1,42	70,28
9	212,5	46,86	y9 = h11 + 1/3 x h9	1,42	66,38
10	906,25	199,83	y10 = h10/2	0,625	124,89
11	1062,5	234,28	y11 = h11/2	0,625	146,43
	2406,25	**530,58**			**407,98**

Source: Author's calculation based on SNI 2833:2016, 2024

Baby factor of heating, K = 1,2
Length of pile cap abutment, By = 9,2 m
The ultimate moment due to the weight of the footing and the landfill
$Mu = K \times M \times B_y$ = 4,504.13 kN.m
Ultimate sliding force due to heavy footing and heaping ground
$Vu = K \times V \times B_y$ = 5.857,5825 kN
Thick abutment walls, Bd = 1,5 m
Number of foundations, N = 3 bh
Arm of the foundation moment against the outer side of the abutment wall, xp = 1.75 m
Maximum moment on the pile cap due to foundation reaction,
$M_p = Pu \times xp \times ny$ = 151,115.03 kN.m
The ultimate moment of the pile cap plan, $M_{your} = M_p - Mu$ = 146,610.91 kN.m
Sliding style of pile cap plan, $V_{your} = x\ P_{max} - Vu$ = 19,944.33 kN
Ultimate moment of plan per meter wide Mu = Mur / By = 5,935.97 kN.m
Plan's sliding force per meter of width $Vu = V_{your} / By$ = 2.167, 86222 kN

a) Reinforcement Bending Pile Cap
The ultimate moment of plan, Mu = 15,935.97 kN.m
Compressive strength of concrete, fc' = 29.05 Mpa
Steel melting stress, f_y = 390 Mpa
Tebal pile cap, h = (hp + ht)/ = 1,75 m
The distance of reinforcement to the outer side of the concrete, d' = 150 mm
Modulus of elasticity of steel, Es = 200,000 Mpa
Concrete stress distribution form factor, b1 = 0.85
$r_b = b1 \times 0{,}85 \times fc'/f_y \times 600 / (600 + f_y) = 0{,}038$
$R_{max} = 0.75 \times \rho_b \times f_y \times [1-0.5 \times 0.75 \times \rho_b \times f_y / (0.85 \times fc')] = 8.673$
Reducing factor of bending strength, $\varphi = 0.8$
Shear force reduction factor, F = 0.6
Thick effective pile cap, d = d - h' = 1,600 mm
The width of the pile cap reviewed, b = 1.000 mm
The nominal moment of the plan, $Mn = Mu / \varphi$ = 19,919.96 kN.m
Moment holding factor, $Rn = Mn \times 106/(b \times d^2) = 7{,}781$
$Rn < R_{max}$ (OK)
Required reinforcement ratio,
$\rho = {,}5 \times fc'/ f_y \times [1 - \sqrt{1} - 2 \times Rn / (0{,}85 \times fc')] = 0{,}02481$
Minimum reinforcement ratio,
$r_{min} = 1{,}4 / f_y = 0{,}00359$
The ratio of reinforcement used, $\rho = 0.00359$
The required area of reinforcement, $As = \rho \times b \times d$ = 5.744 mm2

The diameter of the reinforcement used, D = 32 mm
Required reinforcement distance, s = (π/4 x D ^{2} x b) / As = 140,02 mm
Used reinforcement, L 32 - 125

Spacing between reinforcements is qualified

As = π/4 x D2 x b/s = 6.433.98 mm2

For reinforcement to be taken 20% of the reinforcement of the tree,

As’ = 20% x A_s = 1.286,8 mm2

The diameter of the reinforcement used, D = 16 mm
Required reinforcement distance, s = (π/4 x D ^{2} x b) / As = 156,25 mm
Used reinforcement, L 16 - 150

Spacing between reinforcements is qualified

As' = π/4 x D2 x b/s = 1,340.41 mm2

For the reinforcement of the press shaft 50% of the tensile reinforcement is taken,
As’ = 50% x A_s = 3.216,99 mm2
The diameter of the reinforcement used, D = 25 mm
Required reinforcement distance, s = (w/4 x D2 x b) / As = 152.59 mm
Used reinforcement, L 25 - 150

Spacing between reinforcements is qualified

As' = π/4 x D2 x b/s = 3,272.49 mm2

For reinforcement to be taken 20% of the reinforcement of the tree,

As’ = 20% x A_s = 654,5 mm2

The diameter of the reinforcement used, D = 16 mm
Required reinforcement distance, s = (w/4 x D2 x b) / As = 307.2 mm
Used reinforcement, D 16 - 200

Spacing between reinforcements is qualified

As' = π/4 x D2 x b / s = 1,005.31 mm2

b) Shear Reinforcement Pile Cap
Gaya geser ultimit, Seen = 2,167,862,22 N
Shear strength reduction factor, F = 0.6
Ultimate shearing capacity, V_{cmax} = 5/6 X Φ x (√fc') x b x d = 4.311.844.15 N

Vcmax > V_u (OK), shear safe dimensions

Shear held by concrete, V_c = 1/6 x (√fc'$^{1/2}$) x w x d = 7,746,666.67 N

F x V_c = 4,648,000 N

Φ x Vc > Vu (only need minimum shear reinforcement)

Vs = [Seen – F x V_c] = 2,480,137.78 N

The shear force is held by the shear reinforcement, Vs = 2,480,137.78 N
For shear reinforcement is used diameter, D = 16 mm
The distance of the shear reinforcement in the direction of the y-axis,
and = 400 mm
Shear reinforcement area, A_{and} = w/4 x D2 x w/sy = 502.6548246 mm^2

sx = A_{and} x f_y x d/Vs = 126,47 mm

Used shear reinforcement, D 16 – 100 / 400

The distance between reinforcements is OK

Calculation Result Picture
a) Repeat of the Abutment Foundation

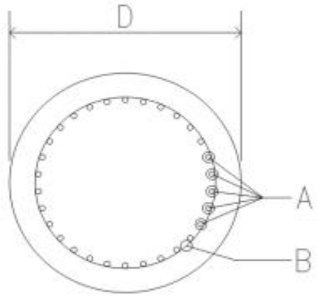


Figure 6. Foundation Repeating
Source: Author's design based on structural analysis, 2024

Foundation dimension data:
Diameter, D = 2,5 m
Depth of foundation, L = 2 m
Foundation's longitudinal recurrence data, 70 D 32
Foundation shear repetition data, 2 D 10 - 75
b) Pile Cap Repeating

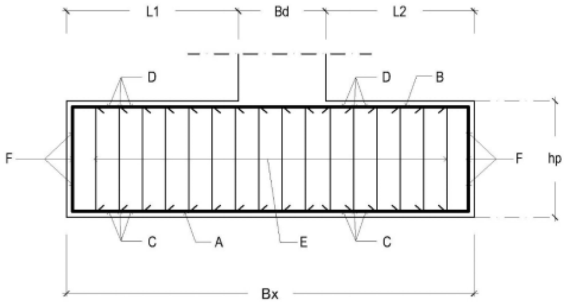


Figure 7. Pile Cap Repeats
Source: Author's design based on structural analysis, 2024

Pile cap dimension data:
Bx = 9,2 m
Bd = 1,5 m
L1 = 3.6 m
L2 = 3.4 m
xp = 1,55 m
a = 1,85 m
hp = 1.75 m
Pile cap recurrence data:

A =	D	32	-	125		
B =	D	25	-	150		
C =	D	16	-	150		
D =	D	16	-	200		
E =	D	16	-	100	/	400
F =	D	32	-	125		

CONCLUSION

Based on the results of the analysis and structural planning of the Tlatar Pabelan Bridge abutment in Central Java, it was concluded that the designed structure met safety requirements for structural loads, overturning and sliding stability, soil bearing capacity, foundation stress, eccentricity, axial forces, and lateral forces in accordance with the applicable design criteria. Furthermore, the reinforcement design for the abutment walls, wing walls, foundations, and pile caps demonstrated that the dimensions and reinforcement specifications were adequate to safely support the structural performance based on the analysis results. To improve the efficiency and accuracy of future design processes, the use of structural analysis software, such as MIDAS or similar applications, is recommended. In addition, the construction of the Tlatar Pabelan Bridge is expected to enhance regional connectivity, facilitate community mobility, and contribute positively to economic development in Central Java and the Special Region of Yogyakarta.

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