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Analysis of Cantilever Retaining Wall as Landslide Mitigation on a Tributary of Cisadane River Bogor City

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ABSTRACT

Bogor City has a high level of rainfall and is included in an area with a high potential for landslides. In March 2021, there was a landslide in a densely populated residential area at RT. 02/03 Cipaku Village, Bogor Selatan District. The landslide occurred on the slopes at the tributary of Cisadane River and adjacent to the nearest connecting bridge. The length of the landslide is 6 meters and as high as 7 meters from the water surface. Mitigation steps to prevent subsequent landslides are needed by constructing a retaining wall, so a cantilever-type retaining wall is chosen by considering the soil parameters and landslide conditions. The results of the analysis show that the stability of the retaining wall against shear (F_s) = 1.04 < SF = 1.5 and for stability against overturning (F_o) = 0.56 < SF = 1.5. Thus, the overall retaining wall is not able to withstand the load of the soil behind it. In terms of stability to the bearing capacity of the soil, the ultimate soil capacity (q_s) = 94.149 kN/m^2 > V = 243.535 kN/m^2. Thus, the subgrade can withstand the load of the retaining wall. Therefore, to meet shear stability and overturning stability, the retaining wall is reinforced by using a bored pile foundation. The results of the foundation analysis showed that the allowable capacity (Q_s) was obtained at 584.8 kN.

Keywords: disaster; mitigation; retaining wall; cantilever; bored pile.

INTRODUCTION

In March 2021, there was a landslide in a densely populated residential area at RT. 02/03 Cipaku Village, Bogor Selatan District The landslide occurred on the slopes at the tributary of Cisadane River and adjacent to the nearest bridge, which connects 2 (two) local communities.

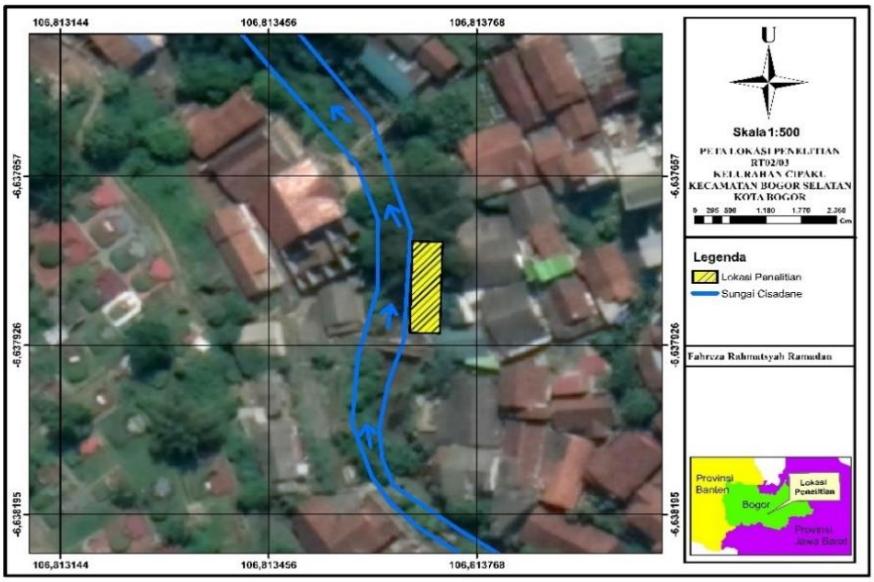

Figure 1. Research Location

The length of the landslide is 6 meters and as high as 7 meters from the water surface, quite dangerous to residents and road users. As relief methods, analysis and design of retaining wall presented in this paper. Previous research on the analysis of retaining walls as landslide mitigation in several cities in Indonesia has been proposed, such as in Southern Lampung, (R. A. Nur et al., 2017), in Magelang, Central Java (Fadhilah & Sudarno, 2017), in Malang (Supriyanto et al., 2017), Bukittinggi (Hakam & Mulya, 2011) and Nangroe Aceh Darussalam (Sadat et al., 2018)

RESEARCH METHOD

This research was carried out in the landslide area along the Cisadane tributary, starting in March 2021. A preliminary survey and field investigation were carried out on March 2, 2021, followed by soil parameters testing in the laboratory. Meanwhile, data processing activities include laboratory data processing with references from (SNI 3420:2016, 2016) and (Badan Standardisasi Nasional, 2017), analysis of retaining wall, and Detail Engineering Design (DED) carried out in April 2021. The research stages are described in the research flow chart, shown in Figure 2.

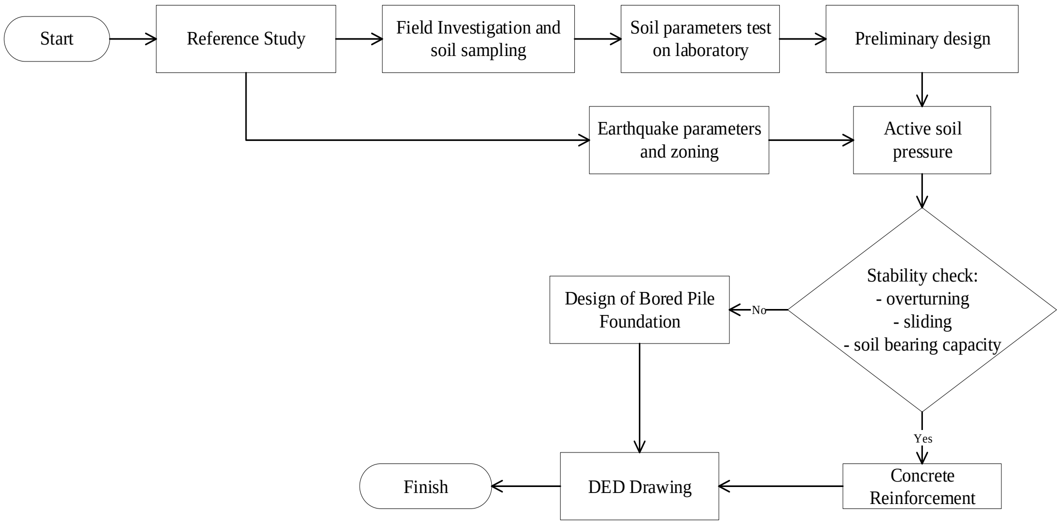

Figure 2. Research Method Flowchart

RESULT AND DISCUSSION

Design of Retaining Wall

The data parameters needed for planning the construction of a cantilever type retaining wall are geotechnical data shown in Table 1, the working load on the wall is shown in Table 2, the dimensions of the plan are shown in Table 3, and seismic data is shown in Figure 3.

Table 1. Soil Parameters

No.	Soil Parameters	Notation	Value	Units
1	Ground water level	MAT	0	m
2	Surface inclanation	i	0	°
3	Friction angle between soil and wall	δ	30,01	°
4	Failure angle	α	90	°
5	Inclanation angle between soil and wall	β	0	°
6	Cohesion	c	53,90	kN/m^2
7	Dry unit weight	γ_{dry}	12,61	kN/m^3
8	Saturated unit weight	γ_{sat}	18,00	kN/m^3
9	Effective unit weight	γ'	8,19	kN/m^3
10	Unit weight of water	γ_w	9,81	kN/m^3

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11	Internal soil friction angle	φ	45,01	^{o}

Table 2. Load Parameters

No.	Load Parameters	Notation	Value	Units
1	Surface load	q	12	kN/m^3
2	Coefficient of Live Load	L	1,6	-
3	Coefficient of Dead Load	D	1,2	-
4	Unit weight of concrete	γ_c	23,52	kN/m^3
5	Gravity acceleration	g	9,81	m/dt^2
6	Vertical earthquake acceleration	a_v	0	g
7.	Horizontal earthquake acceleration	a_h	0,252	g

Table 3. Preliminary design plan

No.	Dimension	Notation	Value	Units
1	Height of Retaining Wall	H	7,5	m
2	Plate foundation thickness	D	0,75	m
3	Crown Width of Retaining Wall	A	0,30	m
4	Plate foundation width	B	3	m

Based on the 2017 earthquake study center map (Pustlitbang PUPR, 2017), Cipaku Village, Bogor Selatan District, Bogor City has a horizontal earthquake coefficient (a) 0.252 g

$$K_h = \frac{Earthquake\ horizontal\ component}{g}$$

$$K_h = \frac{0{,}252 \times 9{,}81}{9{,}81}$$

$$K_h = 0{,}252$$

Thus, inertian angle due to earthquake loads according to the Mononobe-Okabe method can be calculated as follow:

$$\theta = \tan^{-1}\left[\frac{kh}{(1-kv)}\right]$$

$$\theta = \tan^{-1}\left[\frac{0{,}252}{(1-0)}\right]$$

$$\theta = 14.144\,^{\circ}$$

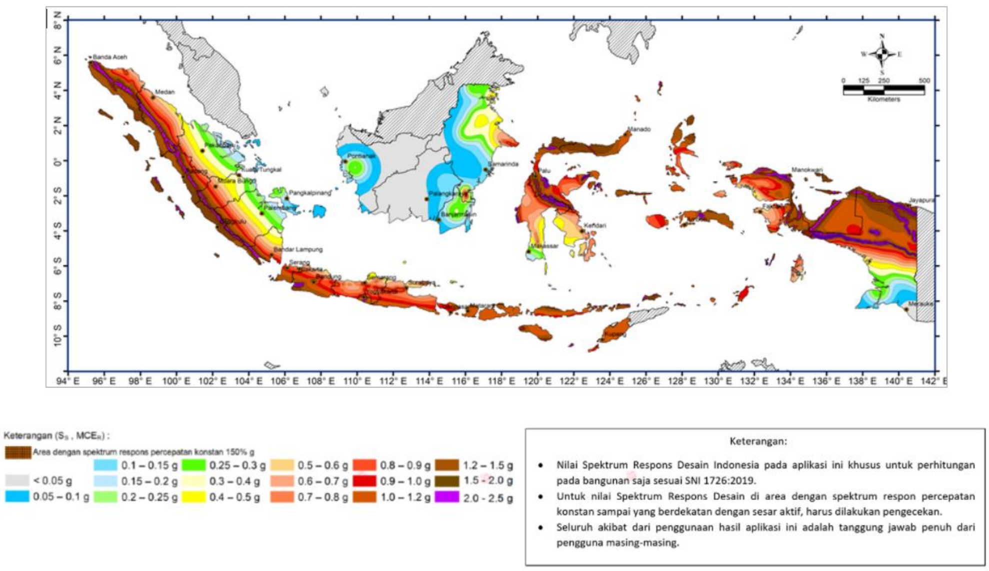

Figure 3. Indonesian Earthquake Zonation Map Source: (Pustlitbang PUPR, 2017)

Calculation of Load and Momen on Wall

To calculate the stability of the cantilever retaining wall, it is necessary to analyze the forces acting on the retaining wall such as active earth pressure and moment forces. The earth pressure is analyzed using Mononobe-Okabe method which calculates the effect of earthquake loads. (O. F. Nur & Hakam, 2010).

$$Ka = \frac{\sin^2(\alpha + \varphi)}{sin^2\,\alpha\ \ sin\,(\alpha\ - \delta)\left\{1 + \sqrt{\dfrac{\sin(\varphi + \delta)\sin(\varphi - \beta)}{\cos(\alpha - \delta)\cos(\alpha + \beta)}}\right\}^2}$$

$$Ka = \frac{0{,}500}{0{,}541\ \times 3{,}565} = 0{,}26$$

The results of the analysis of the active earth pressure acting on the wall are reviewed based on the earth pressure diagram. the active earth pressure diagram is shown in the figure below.

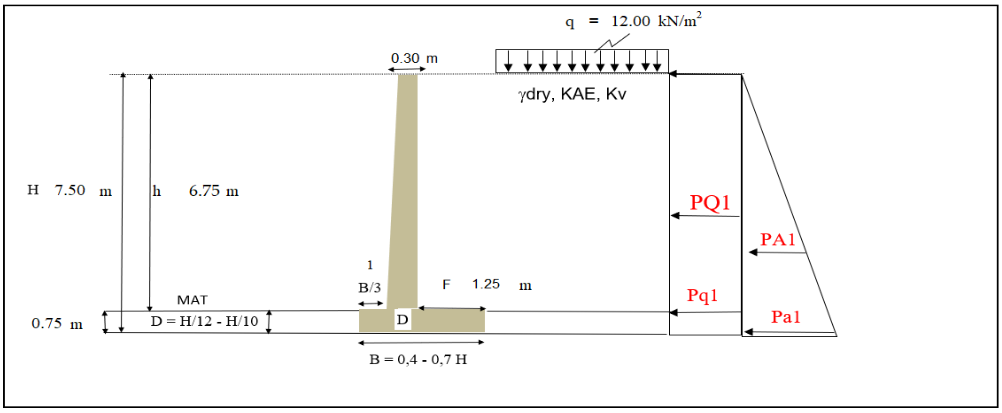

Figure 4. Active Soil Pressure Diagram

The result of lateral active pressure shown on table below.

Table 4. Result of lateral active pressure

Load	P (kN)	Pa_H (kN) $P_{AE}\cos\delta$	Distance from O (m)	Momen (kN.m)
PA_1	82,71	71,70	3,75	268,86
PQ_1	188,93	163,61	2,50	409,01
		ΣPa = 271,72		ΣM = 677,88

Resistance momen (M_R) is calculated based on weight of structure and weight of soil embankment behind the wall. The calculation shown in figure and table below.

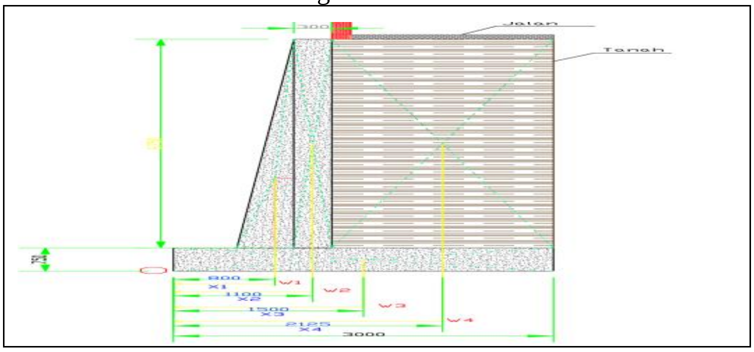

Figure 5 Detail of Load Distribution on Wall

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Table 5 Actual Vertical Momen

Geometric Element	Weight (W) (kN)	Distance from O (m)	Resistance Momen (M_R) (kNm)
W_1	23,81	0,80	19,05
W_2	47,63	1,1	52,39
W_3	52,92	1,5	79,38
W_4	106,42	2,13	226,15
ΣV =	243,535	**ΣM_O =**	376,97

Stabiility Analysis of Cantilever Retaining Wall

The safety of the retaining wall construction structure is reviewed based on the stability to overturning, stability to shear, and stability to the bearing capacity of the soil. (Hardiyatmo, 1993) (Pane et al., 2020)..

Shear Stability

Shear resistance at base of foundation with $d_b = \varphi$:

$$R_h = (\Sigma W + p_{av}) \times tg\ \delta_b$$
$$R_h = (243.535 + 3) \times Tan\ 45{,}01$$
$$R_h = 243{,}62 \text{ kN}$$

Thus, shear stability factor (F_s):

$$F_s = \frac{\sum R_h}{\sum P_h} \geq 1{,}5$$
$$F_s = \frac{243{,}62}{235{,}30}$$
$$F_s = 1{,}04$$

Because F_s = 1,04 < SF = 1,5, then the structure is declared unsafe against shear failure.

Overturning stability Factor (F_o)

$$F_o = \frac{\sum M_R}{\sum M_O} \geq 1{,}5$$
$$F_o = \frac{376{,}971}{677{,}88}$$
$$F_o = 0{,}56$$

Because F_O = 0,56 < SF = 1,5, then the structure is declared unsafe against overturning.

Subgrade Bearing Capacity

Meyerhof method is used to analyze the bearing capacity (Hardiyatmo, 2008), with foundation length factor $s_c = s_q = s_g = 1$, bearing capacity factor N_c = 133,88, N_q = 134,88, $N\gamma$ = 262,74, and foundation width B = B' = 3 meter, then the overburden pressure (P_o) on foundation base is:

$$\text{Po} = Df \times \gamma_b$$
$$\text{Po} = 0{,}75 \times 17{,}85$$
$$\text{Po} = 13{,}39\ kN/m^2$$

Inclanation angle (δ)

$$\delta = arc\ tg\ \frac{H}{V}$$
$$\delta = arc\ tg\ \frac{677{,}877}{376{,}971} = 44{,}02°$$

Inclanation factors (i):

$$i_c = i_q = \left(1 - \frac{\delta}{90}\right)^2 = \left(1 - \frac{44{,}02°}{90°}\right)^2 = 0{,}26$$
$$i_\gamma = \left(1 - \frac{\delta}{\varphi}\right)^2 = \left(1 - \frac{44{,}02°}{30°}\right)^2 = 0{,}0005$$

Depth factors (d):

$$d_c = 1 + 0{,}2 \times \frac{D}{B} \times tg(45° + \frac{\varphi}{2})$$
$$d_c = 1 + 0{,}2 \times \frac{0{,}75}{3} \times tg\left(45° + \frac{45{,}01°}{3}\right) = 1{,}12$$
$$d_q = d_\gamma = 1 + 0{,}1 \times \frac{D}{B} \times tg(45° + \frac{\varphi}{2})$$
$$d_q = d_\gamma = 1 + 0{,}1 \times \frac{0{,}75}{3} \times tg\left(45° + \frac{45{,}01°}{3}\right) = 1{,}06$$

Ultimate bearing capacity (q_u):

$$q_u = s_c\, d_c\, i_c\, c\, N_c + s_q\, d_q\, i_q\, p_o\, N_q + s_\gamma\, d_\gamma\, i_\gamma\, 0.5\, B'\gamma' N_\gamma$$
$$q_u = (1 \times 1{,}12 \times 0{,}26 \times 53{,}90 \times 133{,}88)$$
$$= +(1 \times 1{,}06 \times 0{,}26 \times 13{,}39 \times 134{,}88)$$
$$= +(1 \times 1{,}06 \times 0{,}0005 \times 0{,}5 \times 3 \times 8{,}19 \times 262{,}74)$$
$$= 2612{,}832\ \mathrm{kN/m^2}$$

Nett ultimate bearing capacity ($q_{u\text{-}nett}$):

$$q_{un} = 2612{,}832 - 13{,}39$$
$$q_{un} = 2599{,}45\ kN/m^2$$

Allowable bearing capacity (q_{all}):

$$q_{all} = \frac{q_{un}}{F} + Df\gamma$$
$$q_{all} = \frac{2599{,}45}{32{,}186} + 13{,}39$$
$$q_{all} = 94{,}149\ \mathrm{kN/m^2}$$

Maximum vertical load/m':

$$= q_s \times A_{(per\ meter)}$$
$$= 94{,}149 \times (3 \times 1)$$
$$= 282{,}446\ \mathrm{kN/m^2}$$

Because allowable bearing capacity (q_s) = 94,149 kN/m^2 > V = 243,535 kN/m^2, then the structure is declared safe against general failure.

Global failure

Safety factor for global failure

$$F = \frac{qu\ X\ B'}{q}$$
$$F = \frac{7838{,}497}{243{,}535} = 32{,}186$$

Design of Bored Pile Foundation

Preliminary design for bored hole is shown in table 6.

Table 6. Preliminary design for bored hole

No.	Design Parameters	Notation	Value	Units
1	Bored pile diameter	DM/B	1.00	m
2	Depth of bored pile	D	1.00	m
3	Length of group pile	L	5.00	m
4	Concrete compressive strength	f'c	20,75	MPa
5	Yield strength of steel	fy	300	MPa

Calculation of the stability analysis of the bearing capacity of the bored pile foundation, as group piles

$$Q_g = 2D\ (B + L)\ c + 1.3\ c_b\ N_c\ BL$$
$$Q_g = 2 \times 1 \times (2 + 5)$$

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$= +1{,}30 \times 53{,}90 \times 133{,}88 \times 2 \times 5$
$Q_g = 1754{,}44$ kN
Allowable bearing capacity ($Qg_{(all)}$)= $\frac{1754{,}44}{3} = 584{,}8\ kN > 243{,}53\ kN\ \ SAFE$
Based on the analysis that has been carried out, it can be seen that the stability to shear, stability to overturning, and stability to the bearing capacity of the soil cannot be resisted by the retaining wall itself. Therefore, the moment that causes overturning and the shearing force that occurs in the retaining wall will be resisted by the bored pile system.

Structural Analysis of Retaining Wall

To determine the strength of the structure and determine the need for reinforcement of cantilever retaining walls and bored pile foundations, as well as determine the need for reinforcement for the construction of cantilever retaining walls, the analysis of reinforcement requirements is divided into several pieces. (Hardiyatmo, 2002), shown in Figure below. (Asroni, 2010).

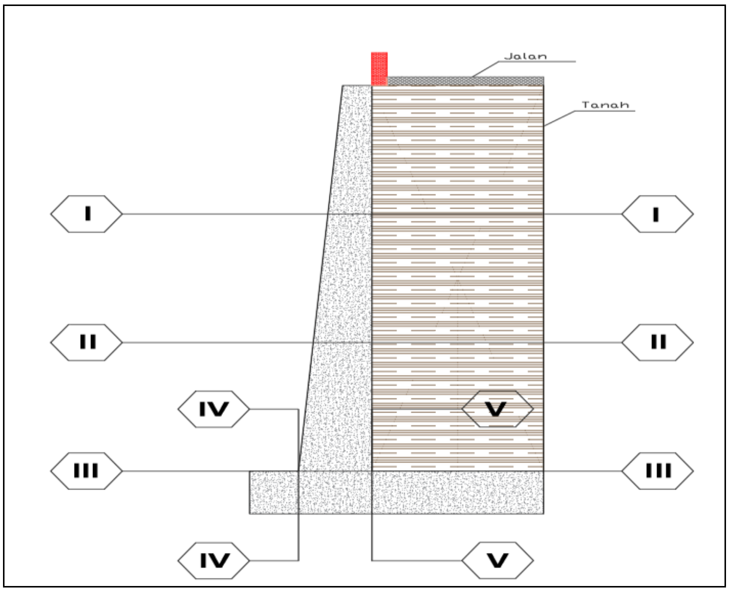

Figure 6. Review point of cantilever retaining wall structure

Shear Force and Moment Analysis

Analysis of shear force and actual moment on vertical wall is shown in Table 7.

Table 7. Actual momen and factored shear force

Slice	y	y^2	y^3	Vu (kN)	Mu (kN)
I-I	2,25	5,06	11,39	17,65	17,44
II-II	4,50	20,25	91,13	48,21	89,12
III-III	6,75	45,56	307,55	91,67	244,07

Retaining wall reinforcement

With $\phi Vn = \phi Vc > Vu$, as shown on table 7, the retaining wall is deemed safe even with minimum reinforcement used. The result of shear reinforcement on slice **II-II** and slice **III-III** shown in table 8 below.

Table 8. Shear reinforcement on retaining wall

Slice	fc' (MPa)	fy (MPa)	b_w (mm)	d (mm)	Vc (kN)	ϕ Vn = ϕ Vc (kN)	Vu (kN)
I-I	24,90	300	1000	434	360,94	270,71	17,65
II-II	24,90	300	1000	534	444,11	333,08	48,21

III-III	24,90	300	1000	734	610,44	457,83	91,67

The calculation results will produce various reinforcement distances on slice II-II, and III-III, as shown on Table 9.

Table 9. Momen reinforcement on vertical wall

Slice	*Y* (m)	Mu (kN.m)	d (mm)	b (mm)	A_s (mm)	n (units)	Design
I-I	2,25	17,44	434	1000	968	4	D16-200
II-II	4,50	89,12	534	1000	1.419	3	D16-200
III-III	6,75	244,07	734	1000	2.160	7	D16-100

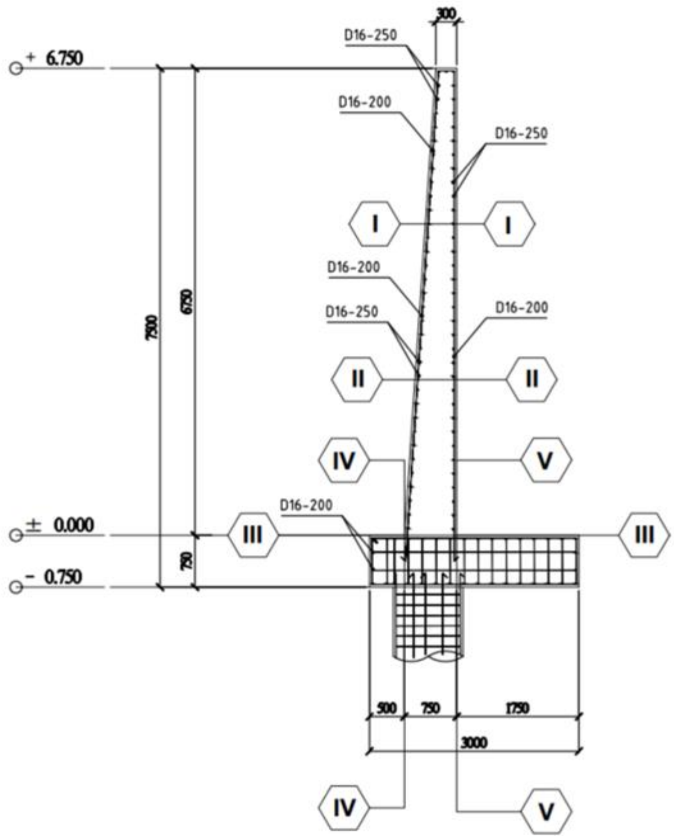

Figure 7. Reinforcement detail on cantilever retaining wall

Reinforcement on Plate-slab

Analysis of shear force and actual moment on plate-slab (slice **V-V**), due to soil pressure on the base of foundation:

$$q = \left(\frac{V}{B}\right) 1 + \left(\frac{6e}{B}\right)$$

$$V = \sum W = 243{,}535$$

$$B = 3{,}00 \text{ m}$$

$q_{max} = 141{,}25$

$q_{min} = 21{,}11$

Dead load factor = 1,2

Live load factor = 1,6

For x = 1 m; $q_2 = \left(\frac{1}{2}\right) \times (94{,}149)$

$= 111{,}97\ kN/m^2$

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For x = 1,25 m; $q_3 = \left(\frac{1}{2}\right) \times (223{,}93)$
$= 111{,}97\ kN/m^2$
Shear force (V_u)
$-\ (94{,}149) \times 0{,}5 \times 1{,}25 = -31{,}29$
$-\ 21{,}11 \times 1{,}25 = -26{,}38$
$+\ (0.75 \times 23{,}52 \times 1{,}2) \times 1{,}25 = 26{,}46$
$+\ (6{,}75 \times 12{,}61 \times 1{,}2) \times 1{,}25 = 127{,}71$
$+\ (6{,}75 \times 1{,}6) \times 1{,}25 = 24{,}00$
$V_u = 120{,}50\ kN$
Momen (M_u)
$-\ (21{,}11) \times 0{,}5 \times 1^2/3 = -16{,}48$
$-\ (71{,}16 - 21{,}11) \times 0{,}5 \times (1{,}25^2/3) = -13{,}04$
$+\ (1{,}25 \times 0{,}75 \times 23{,}52) \times 1{,}2 = 26{,}46$
$+\ (1{,}25 \times) \times 1{,}2 = 127{,}71$
$+\ (1 \times 12) \times 0{,}5 \times 1{,}6 = 28{,}80$
$M_u = 153{,}44\ kN$

With $\phi Vn = \phi Vc > Vu$, as shown above, the base plate is deemed safe even with minimum reinforcement used. The result of shear reinforcement on slice **VI-VI** and slice **V-V** shown in table 10 and Table 11 below.

Table 10. Shear force on base-plate

Piece	fc' (Mpa)	fy (Mpa)	b (mm)	d (mm)	Vc (kN)
VI-VI	24,90	300	1.000	734	610,44
V-V	24,90	300	1.000	734	610,44

Table 11. Shear reinforcement on base-plate

Piece	f Vn = f Vc (kN)	Vu (kN)	As (mm)	Design
VI-VI	457,83	23,94	1468	D16-100
VI-VI	457,83	120,50	878,5	D16-100

Bending Reinforcement for base-plate shown in Table 12.

Table 12. Bending reinforcement on base-plate

Piece	Mu (kN)	As (mm)	n (units)	Design
VI-VI	53,37	1468	7	D16-100
V-V	153,44	878,5	4	D16-100

Reinforcement for *Bored Pile*

In determining the need for bored pile foundation reinforcement, the analysis of reinforcement requirements is divided into several analyzes. The analysis consists of calculations regarding the main reinforcement requirements and the shear reinforcement requirements (Fadli et al., 2021).

Main reinforcement

Effective thickness
Diameter of bored pile foundation = 1.000 mm
$d' = Concrete\ cover + 0{,}50 \times D \times \emptyset$
$d' = 50 + 0{,}50 \times 16 \times 10 = 68\ mm$
$d = D - d'$
$d = 1.000 - 68{,}00 = 932\ mm$

Gross cross-sectional area of foundation
$A_g = 1/4 \times 3{,}14 \times D^2$
$A_g = \frac{1}{4} \times 3{,}14 \times 1.000^2 = 785.000\ mm^2$
Limitaiton of ρ_{min} is 0,0020 (Badan Standardisasi Nasional, 2019), so:

$$A_{s\ needed} = \rho \frac{\pi d^2}{4}$$
$$A_{s\ needed} = 0{,}0020 \times \frac{3{,}14 \times 932^2}{4} = 1363{,}74\ mm^2$$

Main concrete reinforcement needed:
Diameter of reinforcing steel= 16 mm

$$A_{s\ 16} = 1/4 \times 3{,}14 \times 16^2$$
$$A_{s\ 16} = 200{,}96\ mm^2$$

Reinforcement needed:

$$n = \frac{A_{s\ needed}}{A_{s\ (16)}} = \frac{1354{,}94}{200{,}96} = 6{,}79, 7\ units\ used$$
$$A_{st} = 7 \times \frac{1}{4} \times 3{,}14 \times 16^2 = 1406{,}72\ mm^2\ \ 7 - Ø16$$

Minimum axial strength

$$\phi\ Pn\ (maks) = 0{,}85\ \phi\ (0{,}85\ fc'(A_g - A_{st}) + (fy \times A_{st}))$$
$$\phi\ Pn\ (maks) = 0{,}85\ \times 0{,}70\ (0{,}85 \times\ 20{,}75$$
$$= \times\ (785.000 -\ 1.406{,}72)$$
$$= +(300 \times 1.406{,}72))$$
$$\phi\ Pn\ (maks) = 8474372{,}273\ N$$
$$= 8474{,}372\ kN$$
$$\phi\ Pn\ (maks) = 8474{,}372/0{,}7$$
$$= 12106{,}2461\ kN > Pu = 243{,}53$$

Shear reinforcement

Nominal shear strength:

$$V_n = \frac{V_u}{Ø}$$
$$V_n = \frac{120{,}50}{0{,}75} = 160{,}66\ kN$$

Shear resistance of concrete:

$$V_c = 1/6\ \left(1 + \frac{P_u}{14\ A_g}\right)\sqrt{fc'}\,b_w d$$
$$V_c = 1/6\ \times \left(1 + \frac{243{,}535}{14\ \times 785.000}\right) \times\ \sqrt{20{,}75} \times 1.000 \times 932$$
$$V_c = 707592{,}7\ N$$
$$V_c = 707{,}5927\ kN$$

With $\phi Vn= \phi Vc > Vu$, as shown above, the base plate is deemed safe even with minimum reinforcement used. Sectional area of steel reinforcement $A = 1/4 \times 3{,}14 \times 10^2 = 200{,}96\ mm^2$ and spacing $S = 932/2 = 466\ mm^2 \sim 450\ mm^2$. (Ø10-450).

CONCLUSIONS

The following conclusions can be drawn: The dimensions of the construction of the cantilever retaining wall are obtained for a height of 7.5 meters, a footplate width of 3 meters, a footplate height of 0.75 meters, and a peak thickness of a retaining wall of 0.30 meters. The load acting on the retaining wall is 12 kN/m^2, the load is a traffic load with road class III. In addition, the active earth pressure load due to the earthquake was obtained based on the Mononobe-Okabe method, so a total active earth pressure of 271.72 kN was obtained. The earthquake zone obtained is 0.252 g. Stability against shear is obtained at (F_S) = 1.04 < SF = 1.5 and for stability against overturning (F_O) = 0.56 < SF = 1.5, then stability against shear and stability against overturning is deemed unsafe. While the stability of the bearing capacity of the soil obtained a safe ultimate capacity (q_{all}) = 94,149 kN/m^2 > V = 243,535 kN/m^2, the collapse of the bearing capacity of the soil is declared safe. The bored pile

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foundation is designed with a depth of 1 meter and a diameter of 1 meter so that the stability of the bearing capacity of the bored pile foundation in terms of group piles obtained an allowable capacity of 584.8 kN.

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