

Comparative Analysis of Semi Basement Wall With Different Cohesion and Basement Depth in West Jakarta

Luthfy Alfarizi, Resi Aseanto

Civil Engineering of Mercubuana University , Indonesia

LA.alfarizi@gmail.com, resi.aseanto@outlook.com

Abstract

DKI Jakarta has a population density of 19,516 people / square km. Government of Jakarta in the PERDA DKI Jakarta No. 7-2010 CHAPTER 5 Article 42 Paragraph 1 states that "every building must provide parking space". To overcome this, semi-basement will be an option to make a parking space that does not require too much excavation costs. This analysis will compare two types of retaining walls, cantilever retaining walls and conventional retaining walls using the Rankine method for static conditions and the Mononobe-Okabe method for dynamic conditions. This analysis results in the conclusion that for a depth of 2.00m, conventional walls are found to be more efficient in terms of costs with a difference of up to Rp. 178,700 / m'. For soil with a depth of 1.75m, cost efficiency depends on soil cohesion. For the higher soil cohesion, the cost of making cantilever walls is more efficient, reaching Rp. 130,600 / m'. For soil with a depth of 1.50m, with higher cohesion, the cantilevered wall will be more cost-efficient than conventional wall making, reaching Rp. 232,400 / m'.

Keywords

Basement Wall, Cantilever Retaining Wall, Mononobe-Okabe Method, Rankine Method, Semi Basement.

1. Introduction

To make basement, the retaining wall structure have to be able to withstand the active lateral pressures of the soil. In principle, the retaining receives force such as rolling moment, self-weight, active - passive lateral pressure of soil / water, sliding / sliding and uplift. To obtain the best type of retaining wall that can be used for shallow basements in the West Jakarta, it is necessary to compare the strength and costs according to SNI 2847:2013 (2013). To be able to calculate the shape and size of the semi-basement retaining wall suitable for soil in West Jakarta, it is necessary to carry out an analysis of the general description of the shape and size of the soil retaining wall. The shape and size of the walls are influenced by the type of soil in Jakarta and since the soils are not mutual in different areas, this will result in differences in the appropriate shape and different costs of constructing retaining walls.

1.1. Retaining Wall

Retaining walls are structures designed to maintain two different ground elevations (Coduto, 2001).

1.2. Cantilever Retaining Wall

Cantilever retaining wall is a wall consisting of a combination of walls and T-shaped concrete. The thickness of these two parts is relatively thin and fully reinforced to withstand the moment and earth pressure against it.

1.3. Conventional Basement Wall

Conventional basement wall is a concrete wall whose main structure rests and is held together by the lower floor and the upper floor. Soil outside the wall will produce a shear force that is resisted by the lower and upper floor.

1.4. Active Earth Pressure Rankine Method

The minimum lateral soil pressure, which results in shear failure due to the movement of the wall away from the soil behind it is called active soil pressure. Rankine mentions the active earth pressure equation as:

$$P_a = 0,5H^2\gamma K_a \quad (1)$$

with:

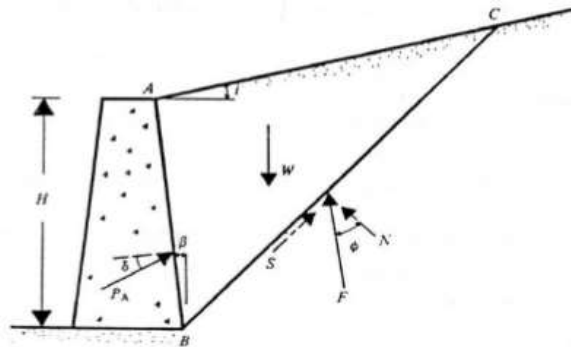
$$K_a: \frac{\sigma_h(active)}{\sigma_v} = \frac{\sigma_3}{\sigma_1} = \frac{1 - \sin \varphi}{1 + \sin \varphi} = \tan^2(45 - \frac{\varphi}{2}) \quad (2)$$

1.5 Active Earth Pressure Mononobe-Okabe Method

The M-O method is an equilibrium-limit approach based on the Coulomb failure field assuming that the walls are moved or turned outward sufficiently to produce a minimum active earth pressure state.

The M-O formulation is expressed as:

$$P_{AE} = (1/2)\gamma H^2(1 - k_v)K_{AE} \quad (3)$$



Picture1 Active Earth Pressure Coloumb

where:

- PAE : Total Lateral Earth Pressure (static + dynamic)
- γ : Soil Weight (kN/m³)
- H : Height of Backfill(m)
- k_v : The Vertical Ground Acceleration.
- KA E : Static Lateral Ground Pressure Plus Dynamic Ground Pressure Coefficient.

$$K_{AE} = \frac{\cos^2(\phi - \theta - \beta)}{\cos\theta \cdot \cos^2\beta \cdot \cos(\delta + \theta + \beta) \left[1 + \frac{\sin(\delta + \phi) \cdot \sin(\phi - \theta - \beta)}{\cos(\delta + \beta + \theta) \cdot \cos(\tau - \beta)} \right]^2} \quad (4)$$

$$\theta = \tan^{-1} \left[\frac{kh}{1 - k_v} \right] \quad (5)$$

1.6 Shifting Stability

The safety factor against shifting (F_{gs}), is defined as:

$$F_{gs} = \frac{\sum R_h}{\sum P_h} \geq 1,5 \quad (6)$$

with:

- $\sum R_h$: The resistance of the retaining wall to displacement
- W : Total weight of the retaining wall and soil over the foundation plate (kN)
- δ_h : Angle of friction between the ground and the foundation
- c : Adhesion between the soil and the base of the wall (kN / m²)
- B : Foundation width (m)
- $\sum Ph$: The sum of the horizontal forces (kN)

1.7. Stability Against Overturning

Lateral ground pressure, caused by soil backfill behind the retaining wall, tends to roll the wall with the center of rotation at the hell end of the foundation plate. The moment of overturning is countered by the moment due to the weight of the retaining wall itself and the moment due to the weight of the soil on the foundation plate. Safe factor against overthrowing, defined as:

$$F_{gl} = \frac{\sum M_w}{\sum M_{gl}} \geq 1,5 \quad (7)$$

$$F_{gl} = \frac{\sum M_w}{\sum M_{gl}} \quad (8)$$

with:

$$\sum M_w : Wb1$$

$$\sum M_w : \text{the moment against the overturning (kN.m)}$$

$$\sum M_{gl} : \Sigma P_{ah} h1 + \Sigma P_{av} B$$

$$\sum M_{gl} : \text{Overturning moment (kN.m)}$$

W : weight of soil above foundation + own weight (kN)

B : the width of foundation

ΣP_{ah} : the number of horizontal forces (kN)

ΣP_{av} : the number of vertical forces (kN)

1.8 Support Capacity

Like other structures, the retaining wall will also settle. Therefore, the basic principles for calculating the settlement of cantilever wall are the same as calculating the settlement of the foundation.

The bearing capacity equation is:

$$F_q : \frac{qu}{q'} \quad (9)$$

with:

qu : q ultimate

qu : $i_c \cdot c \cdot N_c + i_\gamma \cdot 0,5 B' \gamma \cdot N_\gamma$ (10)

i_c, i_γ : load slope factor

N_c, N_q, N_γ : support capacity factor

q' : W/B' (11)

1.9 Reinforcement Bar

To find concrete reinforcement bar, according to the SNI 2847:2013 (2013) are:

As needed : $\rho b d$ (12)

Or

As min : $0,0015 (bw) (d)$ (13)

where

$$\rho : \frac{0,85 f'c}{f_y} \left[1 - \sqrt{1 - \frac{2Rn}{0,85 f'c}} \right] \quad (14)$$

$$Rn : \frac{Mu}{\phi b d} \quad (15)$$

b : concrete width

d : concrete thickness effective

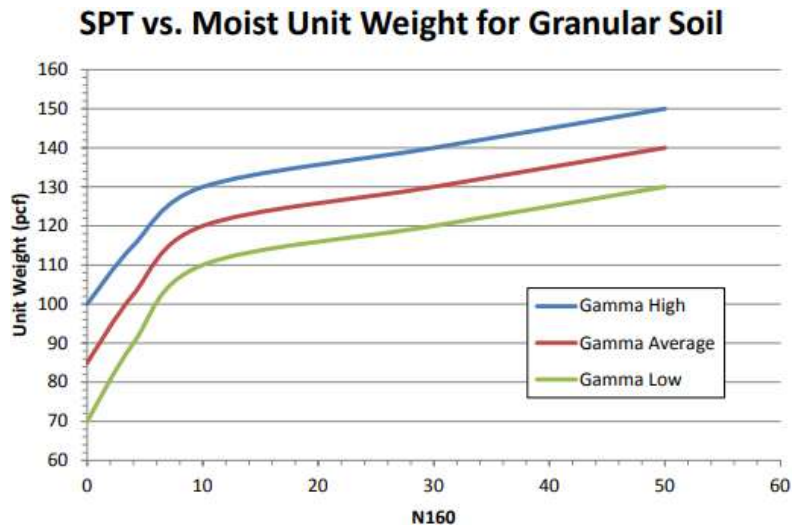


Figure 3. Interpolation of N-SPT Against Soil Weight (Bowless, 1977)

2.5 Conditions to be Calculated

a. Depth of 2.00m

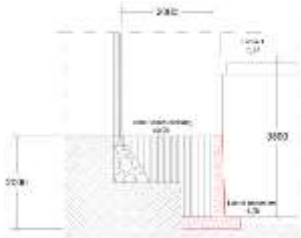


Figure 4. 1a Alternative Design



Figure 5. 2a Alternative Design

b. Depth of 1.75m

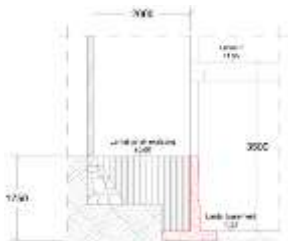


Figure 6. 1b Alternative Design

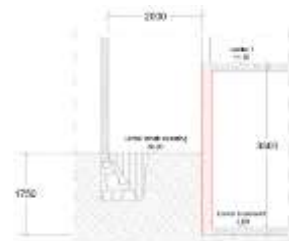


Figure 7. 2b Alternative Design

c. Depth of 1.50m



Figure 8. 1c Alternative Design

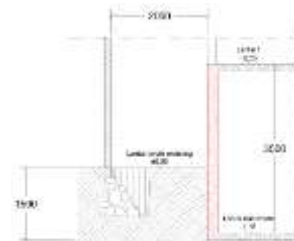


Figure 9. 2c Alternative Design

3. Result and Discussion

3.1. Analysis

In this analysis phase, retaining walls will be analyzed with two types of retaining walls to obtain the strength and dimensions of the walls. It is hoped that after obtaining the calculation of the strength of the retaining wall, we will get the calculation of the cost requirements to construct the retaining wall and make a comparison between the 2 (two) types of retaining walls.

3.2. Soil Parameter

Based on all the data correlations mentioned in the previous stage, the soil parameters for the DKI Jakarta area were obtained as follows.

- Soil Weight (γ_w) = 1,696 t/m³
- Concrete Weight (γ_c) = 2,4 t/m³
- Soil cohesion (c1) = 2 t/m²
- Soil cohesion (c2) = 1,75 t/m²
- Soil cohesion (c3) = 1,50 t/m²
- Soil shear angle (ϕ) = 24°
- Ka (static) = $\text{tg}^2 (45^\circ - 24/2) = 0,4217$
- Bearing capacity factor for longitudinal foundations
- Nc = 19,32
- Nq = 9,60
- N γ = 5,72

3.3. Alternative Design

3.3.1 Static Condition

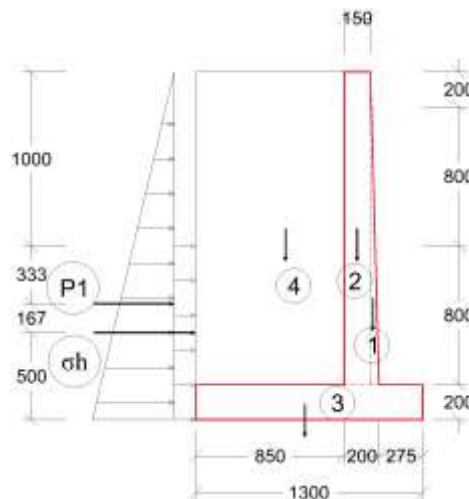


Figure 10. 1a Load Scheme

Table 1. 1a Own Weight

No.	Weight (W) (ton)	Distance from O (m)	Moment to O (ton.m)
1	0,11	0,28	0,03
2	0,65	0,38	0,26
3	0,64	0,66	0,42
4	2,60	0,9	2,33
ΣW	3,99	ΣMw	3,05

$$\sigma_h = \frac{4q}{\pi H} \frac{m^2 n}{(m^2 + n^2)^2} \quad \text{Form} > 0,4$$

$$\sigma_h = \frac{q}{H} \frac{0,203n}{(0,16 + n^2)^2} \quad \text{For } m < 0,4$$

So,
 $\sigma_h = 0,401 \text{ ton}$
 $P_a = 0,5H^2\gamma K_a$
 $P_{a1} = 0,5 (2)^2 (0,421) = 1,43$
 $P_{a2} = (0,401) (1) (0,421) = 0,17$
 $M_{gl} = (1,43) (0,67) + (0,17) (0,50) = 1,04 \text{ t.m}$

3.3.2 Shifting Stability

$$R_h = (2,00) (1,33) + (3,98) (\text{tg } 24^\circ) = 4,42 \text{ t/m}$$

$$F_{gs} = \frac{4,42}{1,60} = 2,76 > 1,50 \dots OK!$$

3.3.3

3.3.3 Overturning Stability

$$F_{gl} = \frac{3,05}{1,04} = 2,93 > 1,5 \dots OK!$$

3.3.4 Dynamic Conditions

$k_v = 0$
 $k_h = 0,365 \text{ g/g} = 0,365$
 $K_{AE} = 0,765$
 $P_{AE1} = (1/2)\gamma H^2(1-k_v)K_{AE}$
 $P_{AE1} = 2,60 \text{ ton}$
 $P_{AE2} = \sigma_h H K_{AE}$
 $P_{AE2} = 0,30 (1) (0,765) = 0,30$
 $M_{gl} = (2,6) (0,67) + (0,30) (0,5) = 1,88 \text{ ton.m}$

3.3.5 Shifting Stability

$$F_{gs} = \frac{4,42}{1,60} = 1,52 > 1,50 \dots OK!$$

3.3.6 Overturning Stability

$$F_{gl} = \frac{3,05}{1,88} = 1,61 > 1,5 \dots OK!$$

3.3.7 Collapse Stability

The calculation is based on the Hansen equation

$$x_e = \frac{3,05 - 1,88}{3,99} = 0,29 \text{ m}$$

$$e = B/2 - x_e = 1,33 \text{ m} / 2 - 0,29 \text{ m} = 0,37 \text{ m} > B/6$$

$$B/6 = 1,36 \text{ m} / 6 = 0,225 \text{ m}$$

$$B' = B - 2e = 1,33 \text{ m} - 2(0,37 \text{ m}) = 0,71 \text{ m}$$

$$A' = B' \times 1 \text{ m}' = 0,71 \text{ m} \times 1 \text{ m}' = 0,71 \text{ m}^2.$$

Load slope factor:

$$i_q = \left[1 - \frac{0,5(2,95)}{3,99 + (0,70)(2,00)(\text{ctg } 24^\circ)} \right]^5 = 0,9965 \geq 0$$

$$i_c = 0,9966 - (1 - 0,9966) / ((19,32)(\text{tg } 24^\circ)) = 0,9963$$

$$i_y = \left[1 - \frac{0,7(2,90)}{(3,99) + (0,70)(2,00)(\text{ctg } 24^\circ)} \right]^5 = 0,9822$$

$$q_u = i_c \cdot c \cdot N_c + i_y \cdot 0,5 B' \gamma N_\gamma$$

$$q_u = 17,05 \text{ t/m}^2$$

$$q' = W/B'$$

$$q' = 5,62 \text{ t/m}^2$$

$$F = \frac{q_u}{q'} = \frac{17,04}{5,62} = 3,03 > 3 (OK!)$$

Or it can also be calculated by:

$$F = \frac{qu(B')}{W} = \frac{17,04(0,71)}{3,99} = 3,03(\text{sama!})$$

3.3.8 Reinforcement Bar

Flexible reinforcement

$$M_u = 2,14 \text{ ton.m}$$

$$t = 200\text{mm}$$

$$d = 120\text{mm}$$

$$R_n = \frac{2,14}{(0,9)(1000)(120)^2} = 1,65$$

$$\rho_{\text{need}} = \frac{0,85(20)}{400} \left[1 - \sqrt{1 - \frac{2(1,65)}{0,85(20)}} \right] = 0,0043$$

$$\text{As need} = \rho b d = (0,0043)(1000)(120) = 520 \text{ mm}^2$$

$$\text{As min} = (0,0015)(1000)(200) = 300 \text{ mm}^2$$

Will be installed D10 - 140mm (As = 549 mm²)

3.3.9 Horizontal Reinforcement

As shrinkage and temperature reinforcement. The minimum required horizontal reinforcement at the base of the wall according to SNI 2847: 2913 Article 14.3.3 is :

$$\text{As min} = 0,0020 (1000) (200) = 400\text{mm}^2$$

0,5 As = 0,5 (400) = 200 mm² and will be installed D10-300mm (As = 235,5 mm²) di both side.

As a vertical reinforcement on the outer wall to support the horizontal reinforcement, reinforcement can be installed D10-300mm.

3.3.10 Design Against Sliding

The critical section for shear views is d = 120mm from the base of the wall. This section is located at 1.80m - 0.12m = 1.68m from the top edge of the wall so that:

$$\text{PAE} = (1,6) (0,5) (\text{KAE}) (\gamma) (\text{H}^2) = (1,6) (0,5) (0,765) (1,696) (1,68)^2 = 2,93 \text{ ton}$$

$$\sigma_h = (1,2)(q)(H)(\text{KAE}) = (1,2)(1,8)(0,68)(0,765) = 0,21 \text{ ton}$$

$$V_u = \text{PAE} + \sigma_h$$

$$V_u = 3,14 \text{ ton}$$

$$\phi V_u = \phi(0,17 \lambda \sqrt{f'c} (b_w)(d))$$

$$\phi V_u = (0,75)(0,17)(1)\sqrt{20}(1000)(120)$$

$$\phi V_u = 68423 \text{ N} = 68,42 \text{ kN} = 6,82 \text{ ton} > V_u \dots \text{OK!}$$

3.3.11 Heel Design

$$V_u = 1,2 [(1,696 \times 0,90 \times 1,80\text{m}) + (2,4 \times 0,9 \times 0,20)] \\ = 4,45 < \phi V_u = 6,82 \text{ ton} \dots \text{OK!}$$

$$M_u = V_u \cdot \frac{1,7}{2}$$

$$M_u = 3,82 \cdot \frac{1,7}{2} = 3,24 \text{ ton.m}$$

$$R_n = \frac{3,24 \times 10^7}{0,9(1000)(120)^2} = 2,50$$

$$\rho_{\text{need}} = \frac{0,85(20)}{400} \left[1 - \sqrt{1 - \frac{2(2,50)}{0,85(20)}} \right] = 0,0068$$

$$\text{As need} = \rho b d = (0,0068)(1000)(120) = 816 \text{ mm}^2$$

$$\text{As min} = \frac{1,4}{f_y} b d = \frac{1,4}{400} (1000)(120) = 420 \text{ mm}^2$$

Will be installed **D10-90mm (As=863mm²)**

In the horizontal direction will be installed **D10-300**

3.3.12 Toe Design

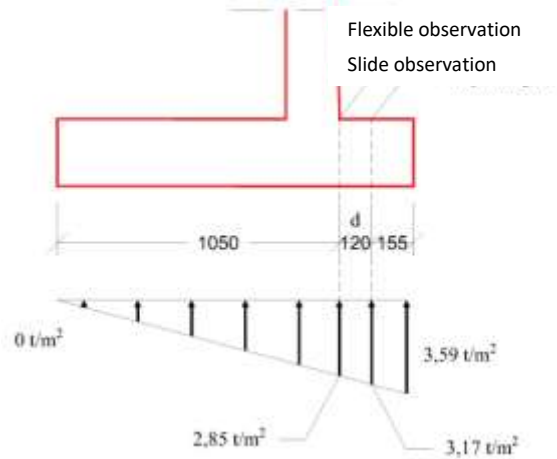


Figure 11. Foundation load distribution

$$M_u = 1,6 [(3,34) (0,25) (0,25/2) + (0,5) (0,25) (3,69-3,34) (2/3) (0,25)] - 1,2 (0,25) (0,20) (0,25/2) (2,4) = 0,17 \text{ ton.m}$$

$$R_n = \frac{0,17}{(0,9)(1000)(120)^2} = 0,12$$

$$\rho_{need} = \frac{0,85(20)}{400} \left[1 - \sqrt{1 - \frac{2(0,12)}{0,85(20)}} \right] = 0,0003$$

$$A_{s \text{ min}} = \frac{1,4}{f_y} b d = \frac{1,4}{400} (1000)(120) = 420 \text{ mm}^2$$

And will be installed D10-160mm ($A_s = 471 \text{ mm}^2$)
 In the horizontal direction will be installed D10-300

3.3.13 Reinforcement Cantilever Wall

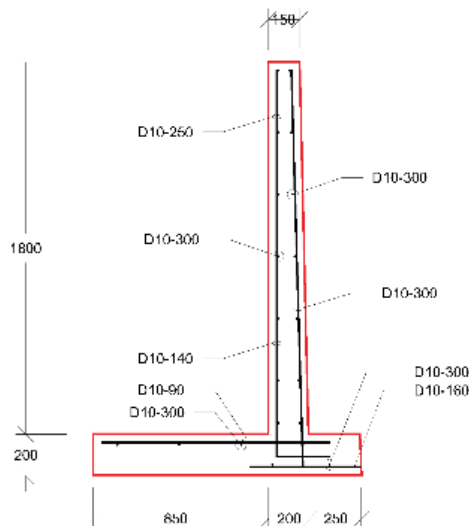


Figure 12. 1a Reinforcement Design

Type 1 Design Alternative
 d. The depth of 1.75m

3.2 Alternative Design

3.2.1 Load Scheme

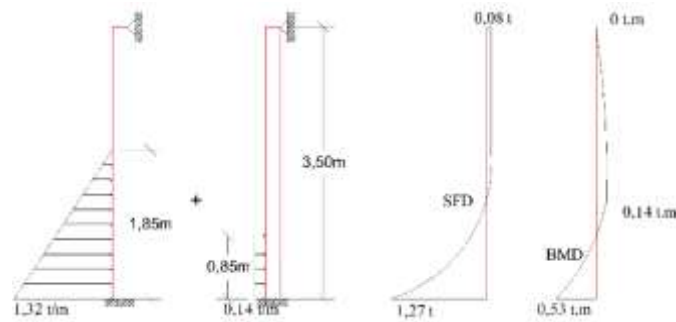


Figure 15. Static load scheme

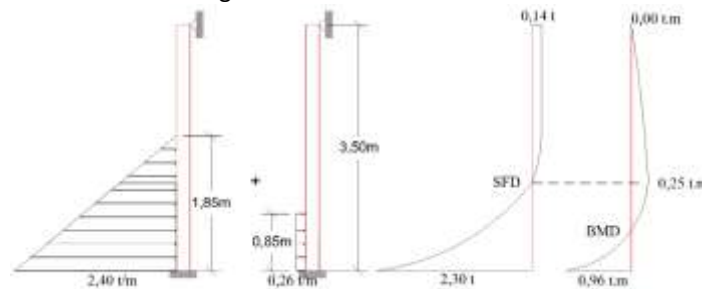


Figure 16. Dynamic load scheme

From the two conditions of the latitude diagram and the bending moment in the figure above, it is obtained:

$$Vu \text{ max} = 1,6 (2,3) : 3,68 \text{ t}$$

$$Mu\text{-max} = 1,6 (0,96) : 1,53 \text{ t.m}$$

$$Mu\text{+max} = 1,6 (0,25) : 0,40 \text{ t.m}$$

3.2.2 Sliding check

The basement walls will be designed with a thickness of 150mm.

So: $d = 150\text{mm} - 75\text{mm}(\text{cover}) - 10\text{mm}(\text{bar diameter}) = 65\text{mm}$.

$$\phi Vc = \phi(0,17\lambda\sqrt{f'c}(b_w)(d)) = 0,75(0,17)(1,0)(\sqrt{20})(1000)(65)$$

$$\phi Vc = 37062\text{N} = 37,06 \text{ kN} = 3,71 \text{ t} > Vu (3,68\text{t}) \dots \text{OK!}$$

3.2.3 Negative bending reinforcement

$$Rn = \frac{Mu}{\phi b d^2} = \frac{1,53 \times 10^7}{0,9(1000)(65^2)} = 4,02$$

$$\rho_{\text{need}} = \frac{0,85 f'c}{fy} \left[1 - \sqrt{1 - \frac{2Rn}{0,85 f'c}} \right] = \frac{0,85(20)}{400} \left[1 - \sqrt{1 - \frac{2(3,75)}{0,85(20)}} \right] = 0,0117$$

$$\text{As need} = \rho b d = 0,0117(1000)(65) = 758,03\text{mm}^2$$

And will be installed D10-100 (785mm²)

3.2.4 Positive bending reinforcement

$$Rn = \frac{Mu}{\phi b d^2} = \frac{0,40 \times 10^7}{0,9(1000)(65^2)} = 1,04$$

$$\rho_{\text{need}} = \frac{0,85 f'c}{fy} \left[1 - \sqrt{1 - \frac{2Rn}{0,85 f'c}} \right] = \frac{0,85(20)}{400} \left[1 - \sqrt{1 - \frac{2(1,04)}{0,85(20)}} \right] = 0,0027$$

$$\text{As need} = \rho b d = 0,0027(1000)(65) = 174,47\text{mm}^2$$

$$A_{s,min} = \frac{1,4}{f_y} b d = \frac{1,4}{400} (1000)(65) = 227,50mm^2$$

And will be installed D10-300 (235,5mm²)

3.2.5 Reinforcement Design

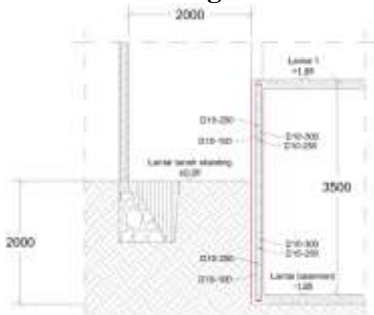


Figure 17. 2a Alternative Design



Figure 18. 2b Alternative Design

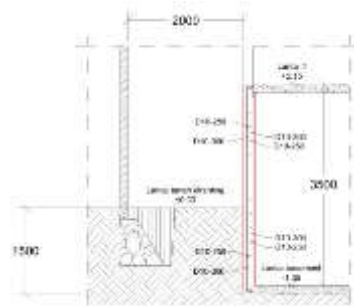


Figure 19. 2c Alternative Design

3.2.6 Alternative 2 Resume

Table 3. Type 2 calculation resume

No.	Depth(m)	Concrete Vol. (m ³ /m')	Reinforcement Vol. (kg/m')
1	2.00	0,5025	47,29
2	1.75	0,5025	38,08
3	1.50	0,5025	34,18

Unit price analysis

Table 4. Unit price of concrete work f'c 20 Mpa

No.	Description	Unit	Index	Unit Price (Rp)	Sum Price (Rp)	
A	Material	Portland Cement	Kg	371	1.360	522.240
		Concrete Sand	m ³	0,494	260.000	128.514
		Gravel	M ³	0,577	290.000	167.394
		Water	Lir	215	3	1.075
B	Man power	Workers	MP	1,65	90.000	148.500
		Stonemason	MP	0,275	100.000	27.000
		Head stonemason	MP	0.028	120.000	3.360
		Foreman	MP	0.083	150.000	12.450
		Total price per unit work				1.012.000

Table 5. Unit price of reinforced bar work per 10kg

No.	Description		Unit	Index	Unit Price (Rp)	Sum Price (Rp)
A	Material	Reinforced Bar	Kg	10,50	12.500	131.250
		Concrete wire	Kg	0,150	22.500	3.375
B	Man power	Worker	MP	0,070	90.000	6.300
		Stonemason	MP	0,070	100.000	7.000
		Head	MP	0,007	120.000	840
		stonemason				
		Foreman	MP	0,004	150.000	600
Total price per unit work						150.000

Table 6. Unit price of excavation work

No.	Description		Unit	Index	Unit Price (Rp)	Total price (Rp)
1	Man	Worker	MD	0,09	90.000	81.000
2	power	Foreman	MD	0,045	150.000	6.750
Total price per unit work						87.750

Table 7. Budget Comparison

No	Depth (m)	Cohesion (t/m^2)	Type 1 (Rp)	Type 2 (Rp)	Difference cost (Type1 – type 2) (Rp)
1		2,00	1.292.200	1.255.100	37.100
2	2,00	1,75	1.352.600	1.255.100	97.500
3		1,50	1.433.800	1.255.100	178.700
4		2,00	1.024.300	1.154.900	-130.600
5	1,75	1,75	1.046.800	1.154.900	-108.100
6		1,50	1.107.900	1.154.900	-47.000
7		2,00	788.800	1.021.200	-232.400
8	1,50	1,75	832.200	1.021.200	-189.000
9		1,50	833.700	1.021.200	-187.500

5. Conclusion And Suggestion

5.1. Conclusion

1. The depth of excavation will affect the shape, volume of concrete and volume of iron required for cantilevered walls because the deeper the excavation, the greater the soil pressure, whereas in conventional walls, the 3 types of depth do not affect the volume of concrete but only affect the volume of iron.
2. Soil cohesion is very influential on the volume of concrete and reinforcement bar volume. The higher soil cohesion, the more economical the cantilever wall will be, while the conventional walls, the soil cohesion has no effect on the volume of concrete nor the iron volume.
3. A suitable retaining wall in West Jakarta is a cantilevered wall for the depths of 1.50m and the depth of 1.75m, while for a depth of 2.00m, conventional walls will be more efficient given the condition in this research.

5.2. Suggestion

1. The calculation uses parameters that are correlated with previous studies so to obtain a more accurate size and reinforcement bar, soil investigation will be needed to determine the condition of existing soil parameters.
2. Cost efficiency can change depending on the depth and distance between the basement floors to the 1st floor, so a careful design is needed to maximize cost efficiency.
3. The distance to the fence and the backfill load design is needed so that we could know more about the difference in wall design.

References

- Coduto, D. P. (2001) *Foundation Design-Principles and Practice*. New Jersey: Prentice-Hall.inc.
SNI 2847:2013 (2013) *Persyaratan beton struktural untuk bangunan gedung*. Badan Standarisasi Nasional (BSN).