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Cantilever Type Retaining Wall Design

Imron Rusadi[1], Era Agita Kabdiyono[2*]

[1,2]Civil Engineering, Faculty of Civil Engineering and Informatics, Dian Nusantara University, Indonesia

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Corresponding author*:
era.agita.k@undira.ac.id

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Abstract: Retaining walls are a crucial soil control technique for mitigating landslides. The slope conditions at the Karawang site are considered unstable. The purpose of this study is to analyze the design of retaining walls at this location using cantilever retaining walls and manual calculations. Data collection techniques include literature review. The analysis results show that the dimensions of the cantilever retaining wall with dimensions of the total height of the DPT = 3.50 m, the total width of the DPT foundation = 3.80 m, the wall thickness = 0.40-0.60 m, the diameter of the main reinforcement used, D = 19 mm, the diameter of the shrinkage reinforcement used, D = 13 mm, the thickness of the concrete cover, s = 40 mm. Based on the soil profile at the location, the soil behind the DPT wall of the original soil does not meet the safety factor, Volume weight (γ1) = 16.7 kN / m3, Internal friction angle, (φ1) = 15.21 º, Cohesion (c1) = 5.7 kN / m2, so it is necessary to replace the granular soil at the back of the DPT to Volume weight (γ1) = 18.5 kN / m3, Internal friction angle (φ1) = 33 º, Cohesion (c1) = 4.16 kN/m2,

Keywords: Retaining Wall, Cantilever Wall, Soil Stability

INTRODUCTION

According to Hardiyatmo (2010), a retaining wall is a structure used to withstand lateral soil pressure caused by fill soil or unstable native soil. This research will discuss the planning of the design of a cantilever type retaining wall, based on soil data obtained in the field,at the planned location in the Karawang area, West Java.

Limitations and scope of the researchRetaining Wall (DPT) PlanningcoveringDPT stability (overturning, sliding, and soil bearing capacity), calculation of moments and shear forces on structural elements as well as reinforcement and crack resistance according to design standards

RESEARCH METHODS

The method used in this research uses Rankine's theory and formula on active soil pressure and passive soil pressure from several modules on soil retaining walls.

The data analysis techniques used in this research are as follows:

1. Planning the type of retaining wall.
2. Carry out planning of the dimensions of the retaining wall.
3. Counting*safety factor*against rolling, sliding and bearing capacity.
4. Calculation of retaining wall structure and reinforcement.
5. Draw the results of the repetition.

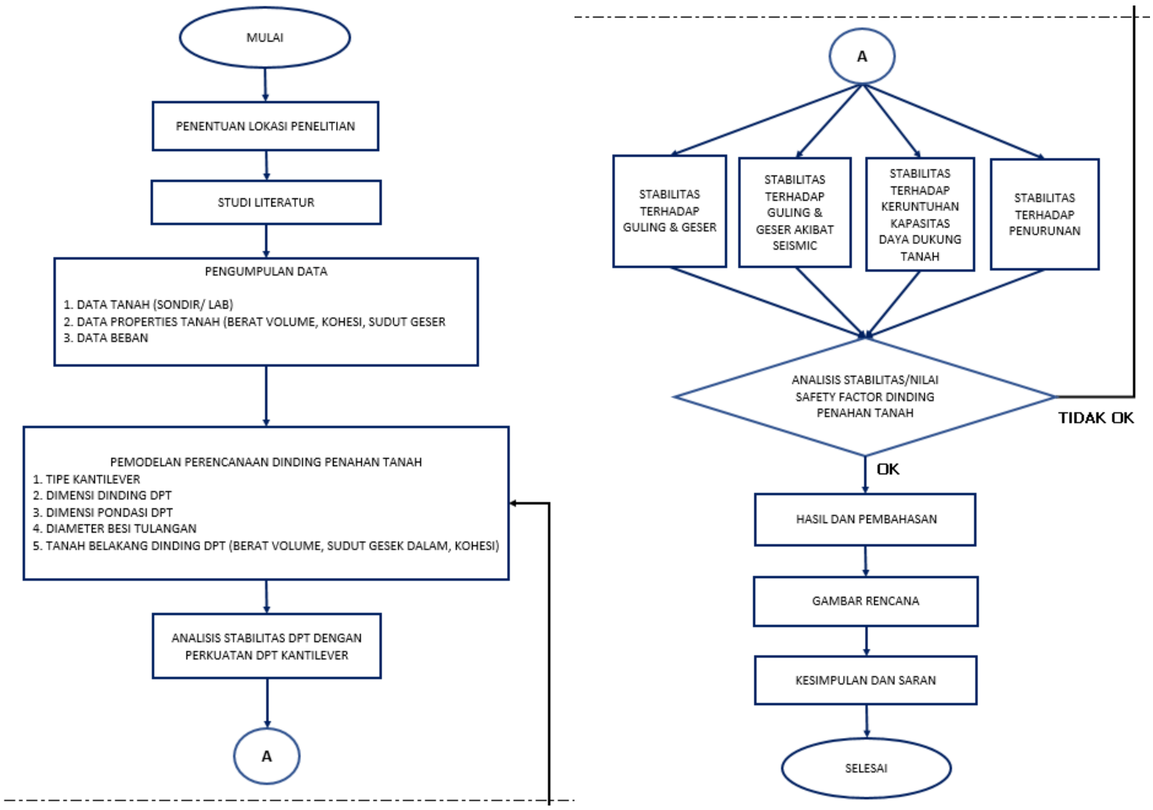


Figure 1. Research Flowchart

RESULTS AND DISCUSSION

Planning Data

Material and Material Data in the DPT Structure

The primary data in this study is land data in Karawang Regency, which will be planned using manual calculations.

Concrete compressive strength	$f_c' = 30$ Mpa
Yield stress of steel for flexural reinforcement	fy = 420 Mpa
Elastic modulus of steel	E_s = 200.000 Mpa
Specific gravity of reinforced concrete	$W_c = 24$ kN/m^3

Reinforced Concrete DPT Dimensional Data

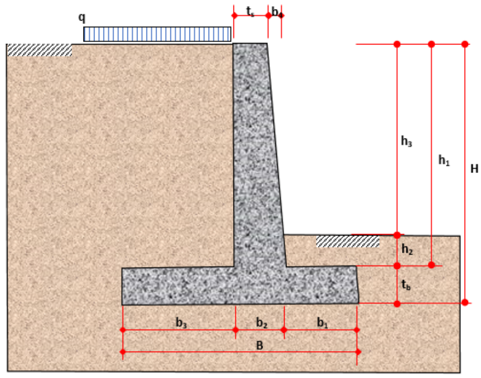


Figure 2. Cantilever Type DPT

H	= 3,50 m,	B	= 3,80 m
t_b	= 0,50 m,	h_1	= 3,00 m
h_1	= 3,00 m,	h_2	= 0,40 m
h_3	= 2,60 m,	t_s	= 0,40 m
b_2	= 0,60 m,	b_1	= 1,20 m
b_3	= 2,00 m,	b_4	= 0,20 m

The main reinforcement used, D = 19 mm
Shrinkage reinforcement used, D = 13 mm
Concrete cover thickness, s = 40 mm

Soil Profile Data at the Location of the DPT wall

The soil behind the DPT wall (native soil does not meet the safety factor)

Volume weight $\gamma_1 = 16{,}7$ kN/m^3

Internal friction angle $\varphi_1 = 15.21\ ^o$

Cohesion $c_1 = 5{,}7$ kN/m^2

The soil behind the DPT wall (replaced with granular soil)

Volume weight $\gamma_1 = 18{,}5$ kN/m^3

Internal friction angle $\varphi_1 = 33\ ^o$

Cohesion $c_1 = 4{,}16$ kN/m^2

Land under the DPT foundation

Volume weight $\gamma_2 = 16{,}7$ kN/m^3

Internal friction angle $\varphi_2 = 15.21\ ^o$

Cohesion $c_2 = 5{,}7$ kN/m^2

Additional Load and Earthquake Load Data

Land site class, SD

Peak bedrock acceleration, PGA = 0.36 g

Vibration amplification factor of period 0 seconds $F_{PGA} = 1$

The burden of living is distributed evenly over the DPT q = 10 kN/m^2

Stability Control on DPT Structure

Calculation of Lateral Forces Acting on DPT

Active soil coefficient

Land behind DPT

$$k_{a1} = \tan(45 - \varphi_1/2)^2 = \tan(45 - 33/2)^2 = 0{,}29$$

Land under DPT

$$k_{a2} = \tan(45 - \varphi_2/2)^2 = \tan(45 - 15.21/2)^2 = 0{,}58$$

Passive soil coefficient

Land behind DPT

$$k_{p1} = \tan(45 + \varphi_1/2)^2 = \tan(45 + 33/2)^2 = 3{,}39$$

Land under DPT

$$k_{p2} = \tan(45 + \varphi_2/2)^2 = \tan(45 + 15.21/2)^2 = 1{,}71$$

Large lateral force due to active soil pressure

$$Pa_1 = 0{,}5.Ka_1.\gamma_1.H^2 = 0{,}5.0{,}29.18{,}5.3{,}5^2 = 33{,}405 \text{ kN/m}$$

The magnitude of the lateral force due to live load

$$Pa_2 = Ka_1 \,.\, q \,.\, H = 0{,}29 \,.\, 10 \,.\, 3{,}5 = 10{,}318 \text{ kN/m}$$

Large lateral force due to passive earth pressure,

$$P_p = 0{,}5.K_{p1}.\gamma_2.(h_2 + t_b)^2 = 0{,}5.3{,}39.16{,}7.(0{,}4+0{,}5)^2 = 22{,}943 \text{ kN/m}$$

Roll and Slide Stability Control on DPT

Roll control on DPT structure

Table 1. Resisting moment on DPT due to structural weight

No	Part	b (m)	h (m)	shape	Burden (kN/m)	Arm (m)	Moment (kNm/m)
1	Wall 1	0,40	3,00	1,00	28,80	1,60	46,08
2	Wall 2	0,20	3,00	0,50	7,20	1,33	9,60
3	Foundation	3,80	0,50	1,00	45,60	1,90	86,64
4	Back Ground	2,00	0,30	1,00	111,00	2,80	310,80
5	Front Land	1,20	0,40	1,00	8,88	0,60	5,33
	Total		S W_t		201,48	S M_p	458,45

Table 2. Overturning moment on DPT due to passive earth pressure

No	Part	b (m)	h (m)	shape	Burden (kN/m)	Arm (m)	Moment (kNm/m)
1	P_p		0,90	1,00	22,94	0,30	6,88
	Total		ΣP_p		22,94	6,88	3,47

Table 3. Overturning moment on DPT due to active earth pressure

No	Part	b (m)	h (m)	shape	Burden (kN/m)	Arm (m)	Moment (kNm/m)
1	P_{a1}		3,50	1,00	33,40	1,17	38,97
2	P_{a2}		3,50	1,00	10,32	1,75	18,06
	Total		S P_a		43,72	S M_g	57,03

Roll control on DPT: Σ Mp / Σ Mg $\geq$ 2.00 □ 8.16 > 2.00 □ SAFE

Slide control on DPT

The resistance force at the base of the DPT due to the weight of the structure and cohesion

$H = c . B + \Sigma W_t . \tan f$ = 5,7 . 3,8 + 201,48 . tan 15,21 = 76,44 kN/m

Passive earth pressure, Pp = 22.94 kN/m

Shear control on DPT conditions with passive earth pressure

Conditions: ΣPp + H / Σ Pa $\geq$ 2.00 □ 2.01 > 2.00 □ AMAN

Shear control on DPT under conditions without passive earth pressure

Conditions: H / S Pa $\geq$ 1,50 □ 1,75 > 1.50 □ SAFE

Overturning and Shear Stability Control on DPT Due to Additional Seismic Loads

Roll control on DPT structure

Elastic earthquake response coefficient at 0 seconds

$A_s = F_{PGA} . PGA = 0.36. 1 = 0.36$ g

Earthquake modification factor, R = 1

Table 4. Overturning moment due to seismic loads on DPT structures

No	Part	b (m)	h (m)	shape	Burden (kN/m)	Arm (m)	Moment (kNm/m)
1	Stem Wall 1	0,40	3,00	1,00	10,37	2,00	20,74
2	Stem Wall 2	0,20	3,00	0,50	7,20	1,50	10,80
3	Foundation	3,80	0,50	1,00	0,00	0,50	0,00
	Total		S W_{ts}		17,57	S M_{gs}	31,57

Horizontal acceleration coefficient,

$K_h = 0,5 . A_s = 0,5 . 0,36 = 0,180$ g

$\theta = \tan^{-1} (K_h) = \tan^{-1} (0,180) = 10,204$ °

Seismic earth pressure,

Seismic active stress coefficient,

$K_{ae} = \cos^2 (\varphi - \Theta) / \cos^2 (\Theta) . \{ 1 + \sqrt{} (\sin \varphi . \sin (\varphi - \Theta) / \cos \Theta)\}$-2

$= \cos^2 (33 – 10,204) / \cos^2 (10,204) . \{ 1 + \sqrt{} (\sin 33 . \sin (33 –10,204) / \cos 10,204)\}$-2 = 0,410

Difference coefficient, $\Delta K_{but} = K_{but} - K_a$ = 0,410 – 0.29 = 0,115

Seismic earth pressure at the top of the structure,

$Q_{eq} = c_1 . H . \Delta K_{but}$ = 18,5 . 3,5 . 0,115 = 7,453 kN/m^2

The magnitude of the shear force due to seismic earth pressure, a

$e_t = 0{,}5 . H . Q_{eq}$ = 0,5 . 3,5 . 7,453 = 13,043 kN/m

Magnitude of moment due to seismic earth pressure,

$M_{ae} = P_{ae}.(2/3 . H)$ = 13,043 .(2/3 . 3,5) = 30,435 kNm/m

Roll control on DPT due to additional seismic loads,

Conditions: $\Sigma Mp / (\Sigma Mg + \Sigma Mgs + Ma) \geq 1.10$ □ 3,85 > 1.10 □ SAFE

Slide control on DPT

Shear control on DPT conditions with passive earth pressure due to additional seismic loads,

Conditions: $H + P_p / (\Sigma P_a + P_{but}) \geq 1.10$ □ 1.75 > 1.10 □ SAFE

Roll control on DPT due to additional seismic loads,

Conditions: $H / (\Sigma P_a + P_{but}) \geq 1.10$ □ 1.35 > 1.10 □ SAFE

Control of Soil Bearing Capacity Against Loads from DPT Structures

Calculation of the forces that occur due to the load from the DPT structure

Table 5. Moments due to the weight of the DPT structure and the soil above the DPT

No	Part	b (m)	h (m)	shape	Burden (kN/m)	Arm (m)	Moment (kNm/m)
1	Wall 1	0,40	3,00	-1,00	28,80	-0,30	-8,64
2	Wall 2	0,20	3,00	-0,50	7,20	-0,57	-4,08
3	Foundation 1	2,00	0,50	1,00	24,00	0,90	21,60
4	Foundation 2	0,60	0,50	-1,00	7,20	-0,40	-2,88
5	Foundation 3	1,20	0,50	-1,00	14,40	-1,30	-18,72
6	Back ground	3,00	3,00	1,00	111,00	0,90	99,90
7	Front Land	1,20	0,40	-1,00	8,88	-1,30	-11,54
	Total		S W_t		201,48	S M_p	75,64

Table 6. Moments due to the influence of seismic loads on the DPT structure

No	Part	b (m)	h (m)	shape	Burden (kN/m)	Arm (m)	Moment (kNm/m)
1	Wall 1	0,40	3,00	-1,00	10,37	2,00	-20,74
2	Wall 2	0,20	3,00	-0,50	2,59	1,50	-3,89
3	Foundation 1	2,00	0,50	-1,00	8,64	0,25	-2,16
4	Foundation 2	0,60	0,50	-1,00	2,59	0,25	-0,65
5	Foundation 3	1,20	0,50	-1,00	5,18	0,25	-1,30
	Total		S W_{ts}		29.38	S M_{gs}	-28,73

Table 7. Overturning moment at DPT for point X

No	Part	b (m)	h (m)	shape	Burden (kN/m)	Arm (m)	Moment (kNm/m)
1	P_{a1}		3,50	-1,00	33,40	1,17	-38,97
2	P_{a2}		3,50	-1,00	10,32	1,75	-18,06
3	P_{a3}		3,50	-1,00	13,04	2,33	-30,43
4	P_{but}		0,90	-1,00	22,94	0,30	6,88
	Total		S Pa		79,71	S Mg	-80,58

The moment arm of the uniform load behind the DPT,

$L= (0.5/ b_3 + b_2 + b_1) - (0,5/ B) = (0,5 /2 + 0,6 + 1,2) - (0,5/ 3,8) = 0,90$ m

Total vertical force on the DPT structure,

$V = \Sigma W_t + q . b_3 = 201,48 + 10 . 2 = 221,48$ kN/m

Total moment force on DPT structure,

$M = \Sigma M_p + \Sigma M_{gs} + \Sigma M_g + q . b_3 . L= 75,64 + (28,73) + (-80,58) + 10 . 2 . 0,9= -15,67$ kNm/m

Foundation Bearing Capacity According to Terzaghi and Peck (1943)

Soil cohesion value below DPT, $c = 5.7$ kN/m2

Depth of DPT base, $D_f = t_b + h_2 = 0,9$ m

The volume weight of the soil at the base of the DPT, $\gamma = 16.7$ kN/m3

Total width of DPT, $B = 3.8$ m

The length of the surveyed DPT, $L = 1$ m

The angle of friction in the soil below the DPT, $\phi = 15,21$ °

$f = f / 180 . \pi = 15,21 / 180 . 3,14 = 0,265$ rad

$e^{(3.\pi / 4 - \phi/2).\tan \phi} = 1,830$

$K_{p\Upsilon} = 3 . \tan^2 [45° + 1/2 . (\phi + 33°)] = 20,584$

Soil bearing capacity factor,

$N_c = 1/ \tan \phi . [a^2 / (2 . \cos^2 (45 + \phi /2) - 1] = 1/ \tan 0,265 . [1,830^2 / (2 . \cos^2 (45 + 0,265 /2) - 1] = 13,027$

$N_q = a^2/[(2.\cos^2 (45+\phi /2)] = N_c . \tan \phi +1 = 1,830^2 / [(2 . \cos^2 (45 + 0,265 /2)] = N_c . \tan 0,265 + 1 = 4,542$

$N_\Upsilon = 1/2 . \tan \phi . [Kp\Upsilon / \cos2 \phi - 1] = 1/2 . \tan 0,265. [20,584 / \cos2 0,265 - 1] = 2,869$

Ultimate bearing capacity of the soil,

$q_u = c . N_c . (1 + 0,3 . B/L) + D_f . \Upsilon . N_q + 0,5 . B . N_\Upsilon . (1 - 0,2 . B/L)$
$= 5,7 . 13,027 . (1 + 0,3 . 3,8/1) + 0,9. 16,7 . 4,542 + 0,5.3,8 . 2,869 . (1 - 0,2.3,8/1) = 228,477$ kN/m^2

Safe number, SF = 3

Permitted soil bearing capacity, $q_a = q_{in} / SF = 228,477 / 3 = 76,159$ kN/m^2

Control of Ground Voltage Occurrence

Eccentricity,

$$e = M / V = (-15.67) / 221.48 = 0.086 \text{ m}$$

Eccentricity control, :

e < B/6 □ 0.09 < 0.63 □ OMG (OK)

The maximum soil stress that occurs at the base of the foundation,

$$q_{max} = V / B. (1 + 6. e / B) = 221,48 / 3,8 . (1 + 6. 0,071 / 3,8) = 66,213 \text{ kN/m}^2$$

The minimum soil stress that occurs at the base of the foundation,

$$q_{min} = V / B. (1 - 6. e / B) = 221,48 / 3,8 . (1 - 6. 0,071 / 3,8) = 50,355 \text{ kN/m}^2$$

Ground voltage control

Conditions: $qa \geq q_{max}$ □ 76,16 > 66,21 □ OMG (OK)

Conditions: $q_{min} > 0.00$ □ 50.36 > 0.00 □ No tensile stress (OK)

REINFORCEMENT IN DPT STRUCTURE

Reinforcement on the DPT wall arm

Internal Force Due to Factored Free Force

Table 8. Combination 1 : 1.2 DL + 1.6 LL + 1.6 H

No	Part	b (m)	h (m)	Load factor	Load (kN/m)	Arm (m)	Moment (kNm/m)
1	P_{a1}		3,00	1.60	53,45	1,00	53,447
2	P_{a2}		3,00	1.60	16,51	1,50	39,621
	Total		S W_{ts}		69.96	S M_g	93,069

The moment of review due to the combination of 1, M_{u1} = 93,069 kN/m

The shear force due to the combination of 1, V_{u1} = 69,956 kN

Table 9. Combination 2 : 1.2 DL + 1.0 LL + 1.6 H + 1.0 EQ

No	Part	b (m)	h (m)	Load factor	Seismic Load (kN/m)	Arm (m)	Moment (kNm/m)
1	Wall 1	0,40	3,00	1.00	10,37	1,50	-15,552
2	Wall 2	0,20	3,00	1.00	7,20	1,00	-7,200
	Total			S W_{ts}	17,57	S M_{gs}	-22,752

No	Part	b (m)	h (m)	Load factor	Load (kN/m)	Arm (m)	Moment (kNm/m)
1	P_{a1}		3,00	1.60	53,45	1,00	-85,516
2	P_{a2}		3,00	1.00	10,32	1,50	-15,477
4	P_{but}		3,00	1.00	13,04	2,00	-26,087
	Total	S W_{ts}			76,81	S M_g	-127,080

The moment of review due to the combination of 2,

$$M_{u2} = (-22,752) + (-127,080) = 149,832 \text{ kN/m}$$

The shear force due to the combination of 2.

$$V_{u2} = 17,57+76,81) = 94,377 \text{ kN}$$

Maximum moment force due to factored load combination,

$$M_{in} = 149,832 \text{ kN/m}$$

Maximum shear force due to factored load combination,

$$V_{in} = 94,377 \text{ kN}$$

Flexural Reinforcement Design

Concrete compressive strength, $fc' = 30$ Mpa

Yield stress of steel for flexural reinforcement, $fy = 420$ Mpa

Distribution shape factor with concrete stress

For : $28 \leq fc' < 55$ MPa,

$\beta_1 = 0,85 – 0,05 . (fc' - 30) / 7 = 0,836$

The shape factor of the concrete stress distribution used, $\beta_1 = 0,836$

Assumption of flexural strength reduction factor, $f = 0,900$

Distance of reinforcement from the outside of the concrete, $d_s = t_s+D/2= 40+19/2= 30,5$mm

Wall thickness, $h = 0.4$ m $= 400$ mm

Effective wall thickness, $h - d_s = 400-30,5 = 369,5$ mm

Considering a 1 m wide wall, $b = 1000$ mm

Reinforcement ratio in balance condition,

$\rho b = \beta 1. 0.85. fc'/ fy. 600 / (600 + fy) = 0.836. 0.85. 30/ 420. 600 / (600 + 420) = 0.030$

Reinforcement ratio at maximum condition,

$\rho_{max} = 0.75 . r_b = 0,75 . 0,030 = 0,022$

Minimum reinforcement ratio,

$\rho_{min} = (0,0018 . 420) / my = (0,0018 . 420) / 420 = 0,0018$

The minimum reinforcement ratio used, $\rho min = 0.0018$

Ultimate moment, $M_{in} = 149,832$ kN/m

Nominal moment of design, $M_n = M_u / \phi = 149,832 / 0,9 = 166,480$ kNm

Moment resistance factor,

$$Rn = Rn = Mn . 10^6 / (b . d^2) = 166,480 . 10^6 / (1000 . 369,5^2) = 1,219$$

Required reinforcement ratio

$\rho = 0.85 . f_c' / f_y . [1 - \sqrt{} [1 – 2 . R_n / (0.85 . f_c')] = 0,85 . 30 / 420 . [1 - \sqrt{} [1 – 2.1,219 / (0,85.30)] = 0,003$

Reinforcement ratio control, $r_{min} < \rho < \rho_{max}$ 0.0018 < 0.0030 < 0.0224 Use ρ

The reinforcement ratio used, $\rho = 0.0030$

Required reinforcement area, $A_s = \rho$ b d = 0.0030. 1000 . 369.50 = 1099.701 mm2

The required reinforcement spacing,

$s = \pi / 4 . D^2 . b / A_s = \pi / 4 . 19^2 . 1000 / 1099{,}701 = 257{,}823$ mm

Maximum reinforcement spacing, $s_{max} = 3 . h = 3 . 400 = 1200$ mm

Maximum reinforcement spacing, $s_{max} = 450$ mm

The reinforcement spacing to be used, s = 257.823 mm

Take the reinforcement distance, s = 100 mm

Reinforcement used, D19-100 mm

Used reinforcement area,

$$A_s = \pi / 4 . D. b / s = \pi / 4 . 19. 1000 / 100 = 2835.287$$

Height of concrete compression block,

$$a = A_s . f_{and} / (0{,}85 * fc' . b) = 2835{,}287 . 420 / (0{,}85 * 30 . 1000) = 46{,}699 \text{ mm}$$

Nominal moment capacity,

$$Mn = A_s . f_{and} . (d - a/2) = 2835{,}287 . 420 . (369{,}50 - 46{,}699/2) = 412{,}203 \text{ kNm}$$

Neutral line height,

$$c = a / \beta 1 = 46.699 / 0.836 = 55.879 \text{ mm}$$

Tensile reinforcement strain,

$$\varepsilon s = (d - c) / c . 0.003 = (369.50 - 55.879) / 55.879 . 0.003 = 0.017$$

Bending reduction factor,

$$\phi = 0.65 \leq 0.65 + (e_s - 0{,}002) / 0{,}003 . 0{,}25 \leq 0{,}9$$

$$= 0{,}65 \leq 0{,}65 + (0{,}017 - 0{,}002) / 0{,}003 . 0{,}25 \leq 0{,}9 = 0{,}90$$

Moment resistance capacity,

$$\phi . Mn = 0{,}90 . 412{,}203 = 370{,}983 \text{ kNm}$$

Control of nominal moment capacity against ultimate moment due to load combination,

Conditions: ϕ .. $Mn \geq Mu$ □ 370.98 > 149.83 □ WATER (OK)

Shrinkage Reinforcement Design

The minimum reinforcement ratio used,

$r_{min} = (0{,}0018 . 420) / f_{and} = (0{,}0018 . 420) / 420 = 0{,}0018$

r_{min} = 0,0014

Required reinforcement area,

$A_s = \rho.\ b.\ d = 0.0014.\ 1000.\ 369.5 = 517.30\ mm^2$

The required reinforcement spacing,

$s = \pi / 4\ .\ D^2\ .\ b / A_s = 3{,}15 / 4\ .\ 13^2\ .\ 1000 / 517{,}30\ = 256{,}59$

Maximum reinforcement spacing, $s_{max} = 3\ .\ h = 3\ .\ 400 = 1200$ mm

Maximum reinforcement spacing, $s_{max} = 450$ mm

The reinforcement spacing to be used, s = 257 mm

Take the reinforcement distance, s = 100 mm

Reinforcement used, D13-100 mm

Used reinforcement area,

$$A_s = \pi / 4\ .\ D.\ b / s = 3.14 / 4\ .\ 13.\ 1000 / 100 = 1327.323$$

Control the area of reinforcement used,:

As ≥ As needed 1327.32 > 517.30 SAFE (OK)

Shear Capacity Check

Shear strength reduction factor, f = 0,75

Effective height of concrete cross-section for shear, d = 369,5 mm

Concrete compressive strength, fc' = 30.0 Mpa

Design ultimate shear force, Vu = 94.377 kN

Concrete shear strength,

$$Vc = 0.17\ .\ \lambda\ .\ \sqrt{fc'}\ .\ b\ .\ d = 0.17\ .\ \lambda\ .\ \sqrt{30}\ .\ 1000\ .\ 369.5 = 344051.924\ N = 344.052\ kN$$

Concrete shear resistance,

$$\phi.\ Vc = 0.75\ .\ 344.052 = 258.039\ Kn$$

Ultimate shear force control on concrete shear resistance,

ϕ . Vc ≥ Vu 258.04 > 94.38 OMG (OK)

Reinforcement on the DPT Rear Foundation

Internal Forces Due to Factored Loads

Table 10. Combination 1 : 1.2 DL + 1.6 LL + 1.6 H

No	Part	b (m)	h (m)	Load factor	Burden (kN/m)	Arm (m)	Moment (kNm/m)
1	Uniform load (q)	2,00	1,00	1,60	32,00	1,00	32,000
2	Back ground	2,00	3,00	1.20	133,20	1,00	133,200
3	Foundation 1	2,00	0,50	1,20	28,80	1,00	28,800
	Total		**S W_t**		**194,00**	**S M_p**	**194,000**

The moment force due to the combination of factored loads, M_{in} = 194,00 kNm

Review moment force due to factored load combination V_{in} = 194,00 kN

Flexural Reinforcement Design

Concrete compressive strength, fc' = 30 Mpa

Yield stress of steel for reinforcement, fy = 420 Mpa

Concrete stress distribution shape factor,

For : $28 \leq fc' < 55$ MPa,

$\beta 1 = 0.85 - 0.05. (fc' - 30) / 7$

$= 0.85 - 0.05 . (30 - 30) / 7 = 0{,}836$

The shape factor of the concrete stress distribution used, $\beta 1 = 0.836$

Assumption of flexural strength reduction factor, $\phi = 0{,}900$

Distance of reinforcement from the outside of the concrete, $d_s = t_s + D/2 = 40+19/2 = 30{,}5$mm

Plate thickness, h = 500 mm

Effective thickness of the plate, $d = h - d_s = 500–30{,}5 = 469{,}5$mm

Considering a 1 m wide plate, b = 1000 mm

Reinforcement ratio under conditions*balance*,

$\rho b = \beta 1. 0.85 . fc'/ fy . 600 / (600 + fy) = 0{,}836. 0.85 . 30/ 420 . 600 / (600 + 420) = 0{,}030$

Reinforcement ratio at maximum condition,

$$\rho_{max} = 0{,}75 . \rho b = 0{,}75 . 0{,}030 = 0{,}022$$

Minimum reinforcement ratio,

$$r_{min} = (0{,}0018 . 420) / f_{and} = (0{,}0018 . 420) / 420 = 0{,}0018$$

$$r_{min} = 0{,}0014$$

The minimum reinforcement ratio used, $\rho_{min} = 0{,}0018$

Ultimate moment, Mu = 194,000 kNm

Nominal design moment, Mn = Mu /f = 194,000 / 0,9 = 215,556 kNm

Moment resistance factor,

$$R_n = M_n .10^6 / (b . d^2) = 215{,}556 .10^6 / (1000 . 469{,}5^2) = 0{,}978$$

Required reinforcement ratio:

$\rho = 0.85 . f_c' / f_{and} . [1 - \sqrt{} [1 – 2 . R_n / (0.85 . f_c')] = 0.85 . 30 / 420 . [1 - \sqrt{} [1 – 2 . 0{,}978 / (0.85 . 30)] = 0{,}002$

Reinforcement ratio control: ρ min < r < ρmax

0,0018 < 0,0024 < 0.0224 □ Use ρ

The reinforcement ratio used, ρ = 0.0024 mm

Required reinforcement area, $A_s = \rho\ b\ d = 0.0024 . 1000 . 469.5 = 1114.941$ mm2

The required reinforcement spacing,

$s = \pi / 4 . D^2 . b / A_s = 3{,}14/ 4 . 19^2 . 1000 / 1114{,}941 = 254{,}299$ mm

Maximum reinforcement spacing, $s_{max} = 3 . h = 3 . 500 = 1500$ mm

Maximum reinforcement spacing, $s_{max} = 450$ mm

The reinforcement spacing to be used, s = 254.299 mm

Take the reinforcement distance, s = 200 mm

Reinforcement used, D19-200 mm

Used reinforcement area,

$$A_s = \pi / 4 . D. b / s = \pi / 4 . 19. 1000 / 200 = 1417.644$$

Height of concrete compression block,

$$a = A_s . fy / (0{,}85 . f_c' . b) = 1417{,}644 . 420 / (0{,}85 . 30 . 1000) = 23{,}349 \text{ mm}$$

Nominal moment capacity,

$$M_n = A_s . fy . (d - a/2) = 1417{,}644 . 420 . (469{,}5 - 23{,}349 /2) = 272{,}594 \text{ kNm}$$

Neutral line height,

$$c = a / b_1 = 23{,}349 / 0{,}836 = 27{,}939 \text{ mm}$$

Tensile reinforcement strain, ε

$$s = (d - c) / c . 0.003 = (469.5 - 27.939) / 27.939 . 0.003 = 0{,}047$$

Bending reduction factor,

$\phi = 0.65 \leq 0.65 + (\varepsilon s - 0.002) / 0.003 . 0.25 \leq 0.9 = 0.65 \leq 0.65 + (0.047 - 0.002) / 0.003 . 0.25 \leq 0.9 = 0.90$

Moment resistance capacity,

$$\phi . M_n = 0.90. 272{,}594 = 245{,}335 \text{ kNm}$$

Control of nominal moment capacity against ultimate moment due to load combination,

Conditions: ϕ . Mn ≥ Mu □ 245,33 > 194.00 □ OMG (OK)

Shrinkage Reinforcement Design

Minimum reinforcement ratio,

$\rho_{min} = (0{,}0018 . 420) / f_y = (0{,}0018 . 420) / 420 = 0{,}0018$

$r_{min} = 0{,}0014$

The minimum reinforcement ratio used, $\rho_{min} = 0{,}0014$

Required reinforcement area, $A_s = \rho\ b\ d = 0.0014 . 1000 . 469.5 = 657.30$ mm2

The required reinforcement spacing,

$s = \pi / 4 . D^2 . b / As = 3{,}14 / 4 . 13^2 . 1000 / 657{,}30 =$ 201,94 mm

Maximum reinforcement spacing, $s_{max} = 3 . h = 3 . 500 = 1500$ mm

Maximum reinforcement spacing, $s_{max} = 450$ mm

The reinforcement spacing to be used, s = 202 mm

Take the reinforcement distance, s = 200 mm

Reinforcement used, D13 – 200 mm

The area of reinforcement used, $As = \pi / 4 . D . b / s = 3.14 / 4 . 13 . 1000 / 200 = 663.661$ mm2

Control the area of reinforcement used, As ≥ As required □ 663.66 > 657.30 □ SAFE (OK)

Shear Capacity Check

Shear strength reduction factor,f = 0,75

Effective height of concrete cross-section for shear, d = 469,5 mm

Concrete compressive strength, fc' = 30.0 Mpa

Design ultimate shear force, V_{in} = 194,00 kN

Concrete shear strength, $V_c = 0.17 . l . \sqrt{f_c'} . b . d = 0{,}17 . \lambda . \sqrt{30} . 1000 . 469{,}5 = 437164{,}759$ N = 437,165 kN

Concrete shear resistance,f . V_c = 0,75 . 437,165 = 327,874 kN

Ultimate shear force control on concrete shear resistance,f . V_c ≥ From □ 327.87 > 194.00 □ AMAN (OK)

Reinforcement on the Front Foundation of DPT

Internal Forces Due to Factored Loads

Table 11. Combination 1 : 1.2 DL + 1.6 LL + 1.6 H

Internal forces in the front foundation of the DPT due to factored loads

No	Part	b (m)	h (m)	Load factor	Burden (kN/m)	Arm (m)	Moment (kNm/m)
1	Front land	1,20	0,40	1,20	9,62	0,60	6,926
2	Foundation 2	1,20	0,50	1.20	17,28	0,60	12,442
	Total			S W_t	26,90	S M_p	19,367

The moment force due to the factored load combination, M_{in} = 19,37 kNm

The moment force due to the factored load combination, V_{in} = 26,90 kN

Flexural Reinforcement Design

Concrete compressive strength, fc' = 30 Mpa

Yield stress of steel for reinforcement, fy = 420 Mpa

Concrete stress distribution shape factor,

For : 28 ≤ fc' < 55 M_{Well}, $\beta 1 = 0.85 - 0.05. (fc' - 30) / 7 = 0.836$

The shape factor of the concrete stress distribution used, $\beta 1$ = 0.836

Assumption of flexural strength reduction factor,f = 0,900

Distance of reinforcement from the outside of the concrete, $d_s = t_s + D / 2 = 40 + 19 / 2 =$ 30,5 mm

Plate thickness, h = 500 mm

Effective thickness of the plate, $d = h - d_s = 500 - 30{,}5 = 469{,}5$ mm

Considering a 1 m wide plate, b = 1000 mm

Reinforcement ratio under conditions*balance*, $r_b = b_1 . 0.85 . f_c'/ f_{and} . 600 / (600 + f_{and}) =$ $0{,}836 . 0.85 . 30/ 420 . 600 / (600 + 420) = 0{,}030$

Reinforcement ratio at maximum condition, $\rho_{max} = 0.75. r_b = 0{,}75 . 0{,}030 = 0{,}022$

Minimum reinforcement ratio, $\rho_{min} = (0{,}0018 . 420) / my = (0{,}0018 . 420) / 420 = 0{,}0018$

$r_{min} = 0{,}0014$

The minimum reinforcement ratio used, $\rho min = 0.0018$

Ultimate moment, $M_{in} = 19{,}367$ kNm

Nominal moment of design,

$$M_n = M_{in} / f = 19{,}367 / 0{,}9 = 21{,}519 \text{ kNm}$$

Moment resistance factor,

$$R_n = M_n . 10^6 / (b . d^2) = 21{,}519 . 10^6 / (1000 . 469{,}5^2) = 0{,}098$$

The required reinforcement ratio,

$\rho = 0.85 . f_c' / f_{and} . [1 - \sqrt{} [1 – 2 . R_n / (0.85 . f_{c'})] = 0.85 . 30 / 420 . [1 - \sqrt{} [1 – 2 . 0{,}098$ $/ (0.85 .30)] = 0{,}0002$

Reinforcement ratio control: $r_{min} < r < r_{max}$

$0{,}0018 < 0{,}0002 < 0{,}0224$ □ Use ρ_{min}

The reinforcement ratio used, $\rho = 0.0018$

The required reinforcement area,

$$As = \rho\, b\, d = 0.0018 . 1000 . 469.5 = 845.100 \text{ mm}^2$$

The required reinforcement spacing,

$$s = \pi / 4 . D^2 . b / A_s = 3{,}14 / 4 . 19^2 . 469{,}5 / 845{,}100 = 338{,}497 \text{ mm}$$

Maximum reinforcement spacing, $s_{max} = 3 . h = 3 . 500 = 1500$ mm

Maximum reinforcement spacing, $s_{max} = 450$ mm

The reinforcement spacing to be used, s = 335.497 mm

Take the reinforcement distance, s = 200 mm

Reinforcement used, D19-200 mm

Used reinforcement area,

$A_s = \pi / 4 . D. b / s = 3.14 / 4 . 19. 1000 / 200 = 1417.644$

Height of concrete compression block,

$a = A_s . f_y / (0,85 . fc' . b) = 1417,644 . 420/ (0,85 . 30 . 1000) = 23,349$ mm

Nominal moment capacity,

$M_n = A_s . f_y . (d - a/2) = 1417,644 . 420 . (469,5 - 23,349 /2) = 272,594$ kNm

Neutral line height,

$c = a / \beta_1 = 23,349 / 0,836 = 27,939$ mm

Tensile reinforcement strain,

$\varepsilon_s = (d - c) / c . 0,003 = (469,5 - 27,939) / 27,939 . 0,003 = 0,047$

Bending reduction factor,

$\phi = 0.65 \leq 0.65 + (e_s - 0,002) / 0,003 . 0,25 \leq 0,9 = 0,65 \leq 0,65 + (0,047 - 0,002) / 0,003 . 0,25 \leq 0,9 = 0,90$

Moment resistance capacity,

$\phi . Mn = 0.90. 272,594 = 245,335$ kNm

Control of nominal moment capacity against ultimate moment due to load combination,

Conditions: $\phi . Mn \geq Mu$ □ 245,335 > 19.37 □ OMG (OK)

Shrinkage Reinforcement Design

Minimum reinforcement ratio,

$r_{min} = (0,0018 . 420) / my = (0,0018 . 420) / 420 = 0,0018$

$r_{min} = 0,0014$

The minimum reinforcement ratio used,

$\rho min = 0,0014$

Required reinforcement area,

$A_s = \rho\ b\ d = 0.0014 . 1000 . 469.5 = 657.30$ mm2

The required reinforcement spacing,

$s = \pi / 4 . D^2 . b / A_s = 3,14 / 4 . 13^2 . 1000 / 657,30 = 201,94$ mm

Maximum reinforcement spacing, $s_{max} = 3 . h = 3 . 500 = 1500$ mm

Maximum reinforcement spacing, $s_{max} = 450$ mm

The reinforcement spacing to be used, $s = 202$ mm

Take the reinforcement distance, s = 200 mm

Reinforcement used, D13-200 mm

The area of reinforcement used,

As = π / 4 . D . b / s = 3.14 / 4 . 13 . 1000 / 200 = 663.661 mm2

Control of the area of reinforcement used: As ≥ As needed □ 663,66 > 657,30 □ SAFE (OK)

Shear Capacity Check

Shear strength reduction factor, f = 0,75

Effective height of concrete cross-section for shear, d = 469.5 mm

Concrete compressive strength, fc' = 30.0 Mpa

Design ultimate shear force, V_{in} = 26,899 kN

Concrete shear strength,

Vc = 0.17 . λ . √ fc' . b . d = 0.17 . λ . √ 30 . 1000 . 469.5 = 437164.759 N = 437.165 kN

Concrete shear resistance,f .

$$V_c = 0,75 \cdot 437,165 = 327,874 \text{ kN}$$

Ultimate shear force control on concrete shear resistance:

ϕ . V_c ≥ From □ 327.87 > 26,90 □ OMG (OK)

CONCLUSION AND SUGGESTIONS

Conclusion

From the calculations and discussions carried out, a conclusion can be drawn:

1. Cantilever type retaining wall with dimensions:

The total height of the wall (H) = 3.5 m

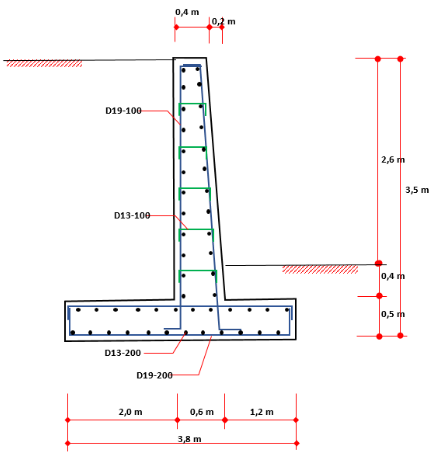
Overall foot plate width = 3.8 m

Vertical wall thickness = 0.4-0.6 m

Height of front (toe) and rear (heel) foot plates = 0.5 m

Front foot plate width (toe) = 1.2 m

Rear foot plate width (heel) = 2.0 m



2. Stability control safety figures obtained:

Roll control on DPT structure

Roll control on DPT = 8.16 > 2.00 □ SAFE

Shear control on DPT condition with active earth pressure = 2.01 > 2.00 □ SAFE

Shear control on DPT condition without passive earth pressure = 1.75 > 1.50 □ SAFE

Overturning and Shear Stability Control on DPT Due to Additional Seismic Loads

Roll control on DPT due to additional seismic load, = 3.85 > 1.10 □ SAFE

Shear control on DPT conditions with passive earth pressure due to additional seismic loads, = 1,75 >1.10 □ SAFE

Roll control on DPT due to additional seismic load, = 1.35 > 1.10 □ SAFE

Control of Ground Voltage Occurrence

Eccentricity control, = 0.09 <0.63 □ OMG (OK)

Ground voltage control

Conditions: qa $\geq q_{max}$ □ 76,16 > 66,21 □ OMG (OK)

Conditions: q_{min}> 0.00 □ 50.36 > 0.00 □ No tensile stress (OK)

REINFORCEMENT IN DPT STRUCTURE

Reinforcement on the DPT wall arm

Control of nominal moment capacity against ultimate moment due to load combination,

Conditions: ϕ. Mn ≥ In □ 370.98 > 149.83 □ OMG (OK)

Control of the area of reinforcement used:

As ≥ As needed □ 1327,32 > 517,30 □ SAFE (OK)

Ultimate shear force control on concrete shear resistance:

Φ . Vc ≥ Vu □ 258.04 > 94.38 □ OMG (OK)

Reinforcement on the DPT Rear Foundation

Control of nominal moment capacity against ultimate moment due to load combination,

Conditions: ϕ . Mn ≥ Mu □ 245.33 > 194.00 □ WATER (OK)

Control of the area of reinforcement used:

As ≥ As needed □ 663,66 > 657,30 □ SAFE (OK)

Ultimate shear force control on concrete shear resistance,

ϕ . V_c ≥ From □ 327.87 > 194.00 □ OMG (OK)

Reinforcement on the Front Foundation of DPT

Control of nominal moment capacity against ultimate moment due to load combination,

Conditions: ϕ. Mn ≥ Mu □ 245.335 > 19.37 □ WATER (OK)

Control of the area of reinforcement used, :

As ≥ As needed □ 663,66 > 657,30 □ SAFE (OK)

Ultimate shear force control on concrete shear resistance:

ϕ . V_c ≥ From □ 327.87 > 26,90 □ OMG (OK)

Suggestion

From the results of the analysis calculations of retaining walls on slopes, several things are recommended, namely:

1. It is necessary to review whether slope stability can be achieved by using other methods.
2. It is necessary to review the use of other types of retaining walls to determine which type is more effective and economical.
3. The calculations for this retaining wall only take into account stability against shear hazards, reinforcement rollover hazards, settlement calculations and soil bearing capacity. If this research is to be used as further research, the researcher can continue to calculate the cost budget plan.
4. In planning, it is necessary to reconsider the soil improvement method, considering that the soil conditions are very poor.
5. In calculating retaining walls, it is hoped that there will be special software for retaining walls so that the calculations for retaining walls can be more precise and efficient.

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