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Evaluation of failure and design of structural reinforcement for gabion-type retaining walls

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ABSTRACT

The geological conditions of South Tangerang are generally alluvium rocks, which consist of clay, silt, sand, gravel, and boulders. This rock type has a good to moderate level of ease of work, the element of resistance to erosion is quite good, therefore the South Tangerang City area is still quite suitable for urban activities. Judging from the distribution of soil types, generally in South Tangerang there are associations of red latosol and reddish brown latosol which are generally suitable for agriculture or plantations. Based on the existing problems, a solution is needed to maintain the stability of the slopes so that subsequent landslides do not occur which can harm the surrounding community, it is necessary to construct a retaining wall to prevent landslides and increase the safety of the occupants, but still maintain the existing structure. The results of this test are used as a construction plan for the gabion type retaining wall. Tests in the laboratory carried out for planning the construction of retaining walls include soil density testing, soil density testing, direct shear strength testing, gradation inspection testing, plastic limit testing, and atterberg limit testing. The dimensions of the cantilever retaining wall construction are 4.5 meters high, the foot plate width is 0.8 meters, the foot plate height is 0.50 meters, and the top thickness of the retaining wall is 0.50 meters. The load acting on the retaining wall is 20 kN/m2. The active earth pressure load due to the earthquake based on the Mononobe-Okabe method, was 143.58 kN. Stability against shear is obtained at (Fgs) = 0.33 < SF = 1.5 and for stability against overturning (Fgl) = 0.45 < SF = 1.5, the stability to shear and stability to roll is not safe. The stability of the bearing capacity of the soil obtained a safe ultimate capacity (qs) = 252.10 kN/m2 > V = 120.0 kN/m2, so that the collapse of the bearing capacity of the soil is declared safe. Bored pile foundation is designed with a depth of 8.0 meters and a diameter of 0.50 meters, with an axial clearance capacity of 578.45 kN, and a lateral capacity of 58.02 kN.

Keywords: retailing wall, test, load acting, axial clearence.

INTRODUCTION

South Tangerang City is a relatively flat area. Some sub-districts have undulating land, such as on the border between Kec. Setu and Kec. Pamulang and some in the district. East Ciputat.

The geological conditions of South Tangerang are generally alluvium rocks, which consist of clay, silt, sand, gravel, and boulders. This rock type has a good to moderate level of ease of work, the element of resistance to erosion is quite good, therefore the South Tangerang City area is still quite suitable for urban activities. Judging from the distribution of soil types,

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generally in South Tangerang there are associations of red latosol and reddishbrown latosol which are generally suitable for agriculture or plantations. However, in reality, more and more people are changing their use for other non-agricultural activities. For some areas such as Kec. Serpong and Kec. Setu, there are soil types that contain sand, especially for areas close to the Cisadane River.

Climatologically, the South Tangerang City area includes a tropical climate with type (Af) which has a high intensity of rainfall, which ranges from 1,800–2,200 mm per year. The air temperature is around 23.4 °C–34.2 °C. The average humidity of the air is 80.0% while the intensity of the sun is 49.0%. The highest rainfall conditions occur in January, which is ±375 mm, while the lowest rainfall conditions occur in July ±75 mm and the average rainfall in a year is 155 mm. The average rainy day per year is 140 rainy days with the highest average rainy day in December of 19 days.

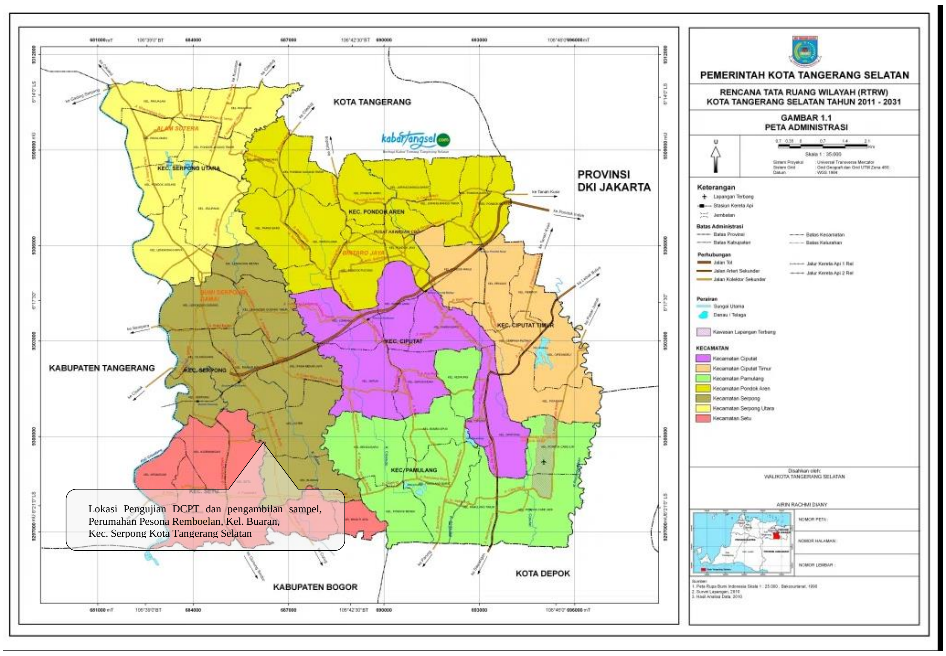

Figure 1 Planning location on the South Tangerang Regional Spatial Plan Map

This has resulted in most areas in South Tangerang City being an area that is prone to landslides. One of the landslides occurred in the Pesona Remboelan Housing Area, which is located on Jl. H. Jamat Gg. Rais, Buaran, Kec. Serpong, South Tangerang City. The gabion type retaining wall located in the housing cluster area of public housing facilities is experiencing movement which is feared to result in material and non-material losses for the residents.

Based on the existing problems, a solution is needed to maintain the stability of the slopes so that subsequent landslides do not occur which can harm the surrounding community, it is necessary to construct a retaining wall to prevent landslides and increase the safety of the occupants, but still maintain the existing structure. The purpose of this study is to provide recommendations for strengthening the structure of the existing retaining wall.

Braja M. Das (1983) describes that the method developed based on the limit state analysis method is the Mononobe-Okabe method (Mononobe and Matsuo, 1929), (Okabe, 1924). The study of the effect of earthquakes on lateral stress in retaining structures was first carried out in Japan by Okabe (1924) and Mononobe-Matsuo (1929). The following is an analysis of the calculation of active earth pressure during an earthquake according to the Mononobe-Okabe method:

PAE= H² (1 – kv) AEC

with:

$$KAE = \frac{\cos^2(\varphi - \theta - \boldsymbol{\beta})}{\cos\theta \cos^2 \boldsymbol{\beta} \,\mathbf{cos}\,(\delta + \boldsymbol{\beta} + \theta)\left\{1 + \left[\sqrt{\dfrac{\sin(\delta+\varphi)\sin(\varphi - \theta - \mathbf{i})}{\cos(\delta + \boldsymbol{\beta} + \theta)\cos(\mathbf{i} - \boldsymbol{\beta})}}\right]^{\frac{1}{2}}\right\}^2}$$

Planning of retaining wall construction needs to be considered on several factors so that the construction remains safe. Based on SNI 8460:2017, retaining walls must be designed to remain secure against shear stability, stability against overturning, and stability against collapse of the bearing capacity of the soil.

RESEARCH METHODS

Field testing in this study was in the form of handboring and field density testing which was carried out to obtain original soil parameter data. The original soil parameters were then brought to the soil mechanics laboratory for testing of physical and mechanical properties. The results of this test are used as a construction plan for the gabion type retaining wall.

Tests in the laboratory carried out for planning the construction of retaining walls include soil density testing, soil density testing, direct shear strength testing, gradation inspection testing, plastic limit testing, and atterberg limit testing.

ANALYSIS RESULTS AND DISCUSSION

Safety Overview of Existing Ground Retaining Wall
Mononobe-Okabe Method

Planning for retaining wall structures requires data parameters to plan it. The data parameters needed to plan the construction of a cantilever type retaining wall are geotechnical data, the load acting on the retaining wall, the dimensions of the retaining wall plan, and seismic data. The geotechnical data in table 1 was obtained from the results of soil investigations in the field and in the soil mechanics laboratory.

Table 1 Geotechnical data

No.	Geotechnical Data	Notation	Value	Unit
1	Ground water level	MAT	1	m
2	Ground level slope	i	0	o
3	The angle of friction between the wall and the ground	d	20,67	o
4	Land failure slope angle	a	90	o
5	The angle of inclination of the wall to the ground	b	14	o
6	soil cohesion	c	20	kN/m^2
7	Dry soil volume	g_{dry}	16	kN/m^3
8	Saturated soil volume	g_{sat}	18	kN/m^3
9	Effective soil volume	g'	6,7	kN/m^3
10	Weight of water	g_w	9,81	kN/m^3
11	Friction angle in the ground	φ	22	o

Source: Test results

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Table 2 Loads acting on retaining walls

No.	Loading	Notation	Value	Unit
1	Fasum evenly distributed load	q	20	kN/m^3
2	Live load	L	1,6	-
3	Dead load	D	1,2	-
4	Fit weight. gabions	g_c	20	kN/m^3
5	Gravitational acceleration	g	9,81	m/dt^2
6	Vertical earthquake acceleration	a_v	0	g
7.	Horizontal earthquake acceleration	a_h	0,15	g

Source: Calculation results

Table 3 Dimensions of the Gabion type retaining wall plan

No.	Planning Dimension	Notation	Value	Unit
1	Retaining wall height	H	4,5	m
2	Foundation thickness	D	1,0	m
4	Peak Width	A	0,50	m
5	Width of foundation base	X	0,80	m

Source: Calculation results

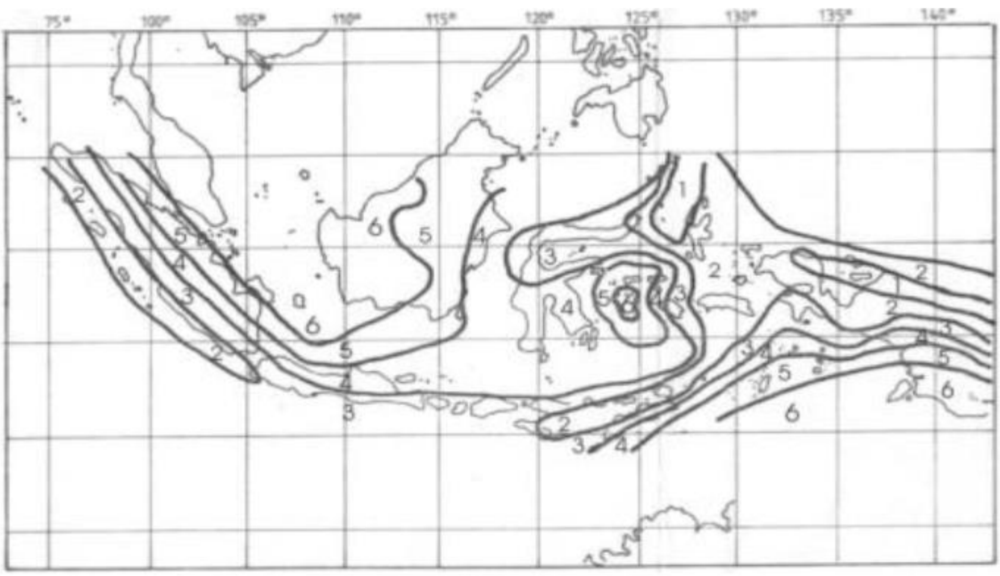

Figure 2 Map of Indonesia's earthquake zone

Based on Kimpraswil guidelines No: Pt T-10-2002-B, Kec. Serpong, South Tangerang City is included in zone 4, so the horizontal earthquake coefficient is obtained:
Zone 4 = 0.15 g

$$K_h = \frac{\text{horizontal component of the earthquake direction}}{g}$$

$$K_h = \frac{0,15 \times 9,81}{9,81}$$

K_h = 0,15

Therefore, the angle of inertia due to earthquake loads according to the Mononobe-Okabe method:

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

$\theta = \tan^{-1}\left[\frac{0{,}15}{(1-0)}\right]$

$\theta = 8{,}531\ ^{\circ}$

As a step in analyzing the stability of the retaining wall structure, it is necessary to identify the forces acting on the retaining wall such as active earth pressure and moment forces. The coefficient of active earth pressure on the wall is used the Mononobe-Okabe method which takes into account the effects of earthquake loads.

$$KAE = \frac{\cos^2(\varphi - \theta - \boldsymbol{\beta})}{\cos\theta\cos^2\boldsymbol{\beta}\,\mathbf{cos}\,(\delta + \boldsymbol{\beta} + \theta)\left\{1 + \left[\sqrt{\frac{\sin(\delta+\varphi)\sin(\varphi - \theta - \mathbf{i})}{\cos(\delta + \boldsymbol{\beta} + \theta)\cos(\mathbf{i} - \boldsymbol{\beta})}}\right]^{\frac{1}{2}}\right\}^2}$$

$KAE = \frac{1}{0{,}603 \times 2{,}635}$ = 0,511

The results of the analysis of the active earth pressure acting on the wall are reviewed based on the earth pressure diagram.

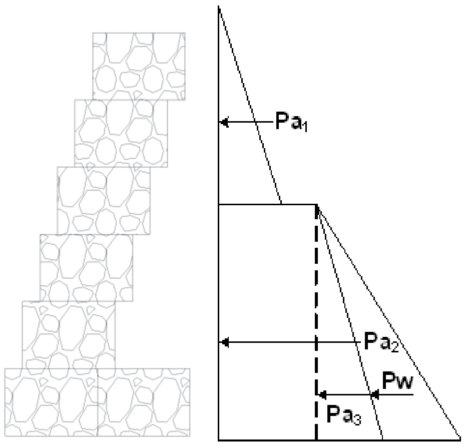

Figure 3. Active earth pressure diagram Source: Planning Drawing

Results of active earth pressure analysis:

Table 4 Active earth pressure

Style	P (kN)	PaH (kN) PAE x Cos d	Distance from O (m)	Moment to O (kN.m)
P_A1	8,46	9,729	3,50	34,05
P_A2	33,84	38,916	1,50	58,37
P_A3	17,32	19,895	1,00	19,89
P_W	44,1	50,715	1,00	50,71
P_Q1	21,15	24,3225	4,50	109,45
	ΣP_{AE} = 143,58			ΣM = 272,48

The safety of retaining wall construction structures is reviewed based on stability against overturning, stability against shearing, and stability against soil bearing capacity.

Shear resistance assuming that db = φ, namely:

$$R_h = (\Sigma W + p_{av}) \times tg\,\delta_b$$
$$R_h = (120{,}0 + 0) \times \text{Tan } 22$$
$$R_h = 48{,}48 \text{ kN}$$

Stabilitas geser didapatkan:

$$F_{gs} = \frac{\sum R_h}{\sum P_h} \geq 1{,}5$$
$$F_{gs} = \frac{48{,}48}{143{,}58}$$
$$F_{gs} = 0{,}33$$

because Fgs = 0.33 < SF = 1.5, then shear stability is not safe.
The calculation of overturning stability is obtained:

$$F_{gl} = \frac{\Sigma M_w}{\Sigma M_{gl}} \geq 1{,}5$$
$$F_{gl} = \frac{132{,}96}{272{,}48}$$
$$F_{gl} = 0{,}448$$

because Fgl = 0.21 < SF = 1.5, stability against overturning is not safe.
Analysis of the stability of the bearing capacity of the soil is used by Meyerhof (1963), the elongated foundation factor sc = sq = sg = 1, the bearing capacity factor Nc = 32.67, Nq = 20.63, Ng = 18.56, and the foundation width B = B' = 2 meters.
Overbuden pressure at the base of the foundation:

$$Po = Df \times \gamma_b$$
$$Po = 0{,}80 \times 18{,}0$$
$$Po = 14{,}40 \text{ kN/m}^2$$

The resultant inclination angle of the load to the vertical direction:

$$\delta = \text{arc tg } \frac{H}{V}$$
$$\delta = \text{arc tg } \frac{143{,}58}{120{,}0} = 50{,}12°$$

Load tilt factor:

$$i_c = i_q = \left(1 - \frac{\delta}{90°}\right)^2 = \left(1 - \frac{50{,}12°}{90°}\right)^2 = 0{,}19$$
$$i_\gamma = \left(1 - \frac{\delta}{\varphi}\right)^2 = \left(1 - \frac{50{,}12°}{22°}\right)^2 = 1{,}63$$

Depth factor:

$$d_c = 1 + 0{,}2 \times \frac{D}{B} \times tg(45° + \frac{\varphi}{2})$$
$$d_c = 1 + 0{,}2 \times \frac{0{,}8}{1} \times tg\left(45° + \frac{22°}{2}\right) = 1{,}48$$
$$d_q = d_\gamma = 1 + 0{,}1 \times \frac{D}{B} \times tg(45° + \frac{\varphi}{2})$$
$$d_q = d_\gamma = 1 + 0{,}1 \times \frac{0{,}8}{2} \times tg\left(45° + \frac{22°}{2}\right)$$
$$= 1{,}06$$

Ultimate bearing capacity:

$$q_u = s_c\, d_c\, i_c\, c\, N_c + s_q\, d_q\, i_q\, p_o\, N_q + s_\gamma\, d_\gamma\, i_\gamma\, 0.5\, B'\gamma' N_\gamma$$
$$q_u = (1\, x\, 1{,}48\, x\, 0{,}19\, x\, 20\, x\, 20{,}27)$$
$$= +(1 \times 1{,}06 \times 0{,}19 \times 1 \times 14{,}4 \text{ x } 9{,}19)$$
$$= +(1 \times 1 \times 1{,}63 \times 0{,}5 \times 1{,}6 \times 6{,}3 \times 5{,}09)$$
$$= 182{,}47 \text{ kN/m}^2$$

Net ultimate bearing capacity:

$q_{un} = 182{,}47 - 14{,}40$
$q_{un} = 168{,}07 \mathrm{kN/m^2}$
Permit bearing capacity:

$$q_i = \frac{q_{un}}{F}$$

$$q_s = \frac{168{,}07}{2}$$

$q_s = 84{,}03 \ \mathrm{kN/m^2}$
Maximum total vertical load on the base of the foundation per meter of length:
$= q_s \times \text{area per meter of length}$
$= 84{,}03 \times (3 \times 0{,}8)$
$= 252{,}10 \ \mathrm{kN/m^2}$
because, qs = 252.10 kN/m2 > V = 120.0 kN/m2, then the collapse of the soil bearing capacity is declared safe.
Simplified Bishop Method
By using the Hyrcan application ver. 1.7, it is shown that the value of the Factor of Safety (FK) of the slope with the Simplified Bishop method has a value of 0.64. Thus, it can be stated that the slope is unstable and can experience a slide at any time

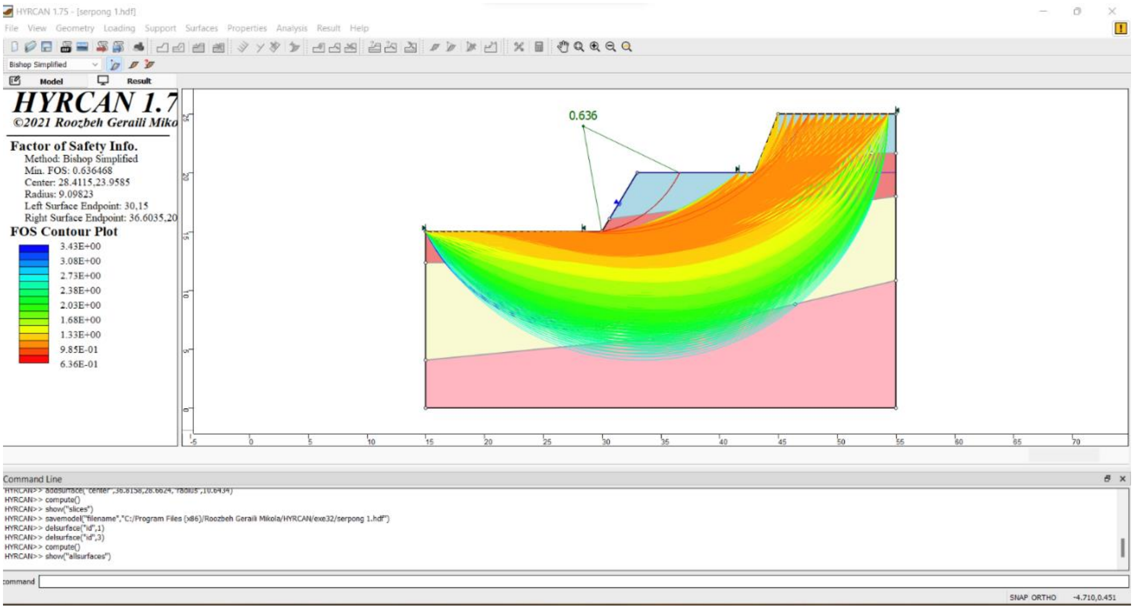

Figure 4. The local safety factor of the downslope

By using the Hyrcan application ver. 1.7, it is shown that the global Safety Factor (FK) value with the Simplified Bishop method on slopes has a value of 0.89. Thus, it can be stated that the slope is unstable and can experience a slide at any time.

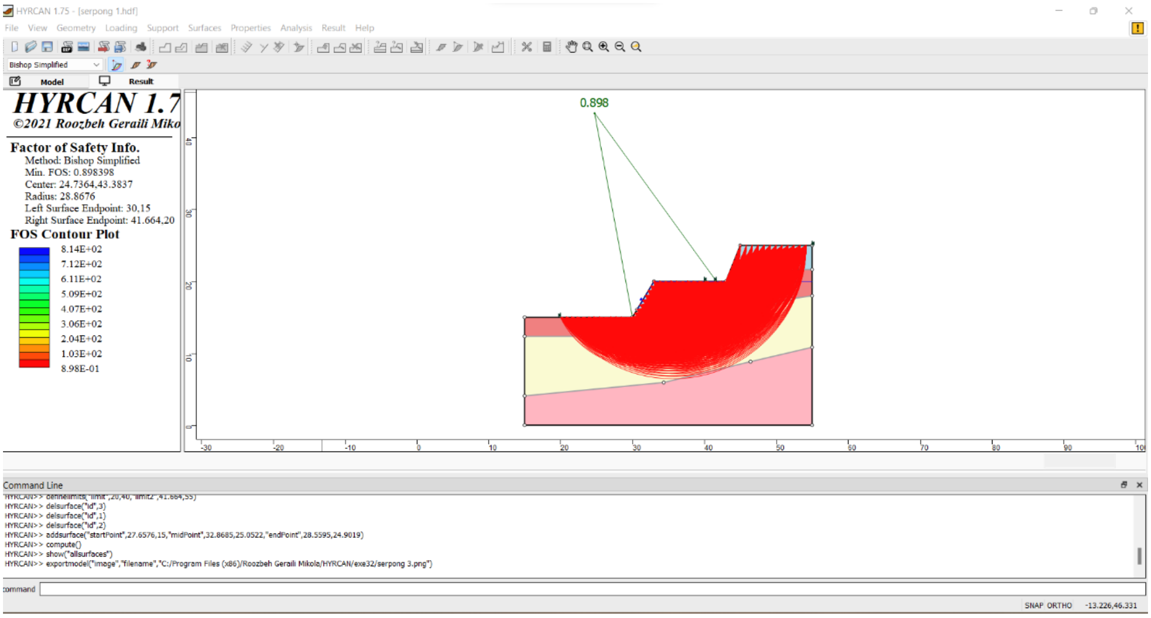

Figure 5. The global safety factor of the slope

Finite Element Method (FEM)

In the figure below, the finite element mesh modeling of the existing slope that has been reinforced with a gabion type retaining wall is shown. Modeling is done with the Adonis application ver. 3.25.

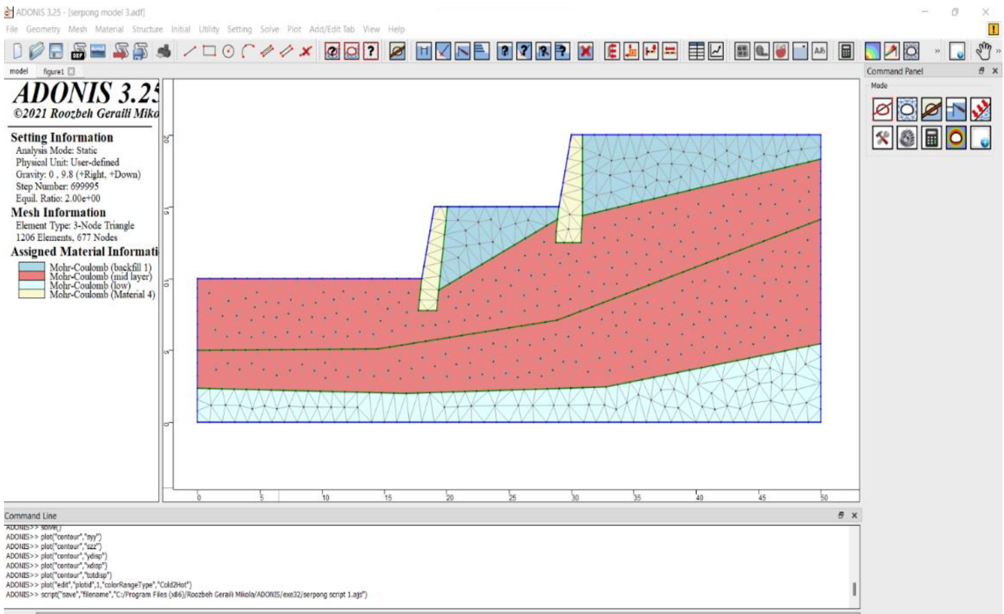

Figure 6 Finite element mesh model

The modeling results are shown in the pictures below.

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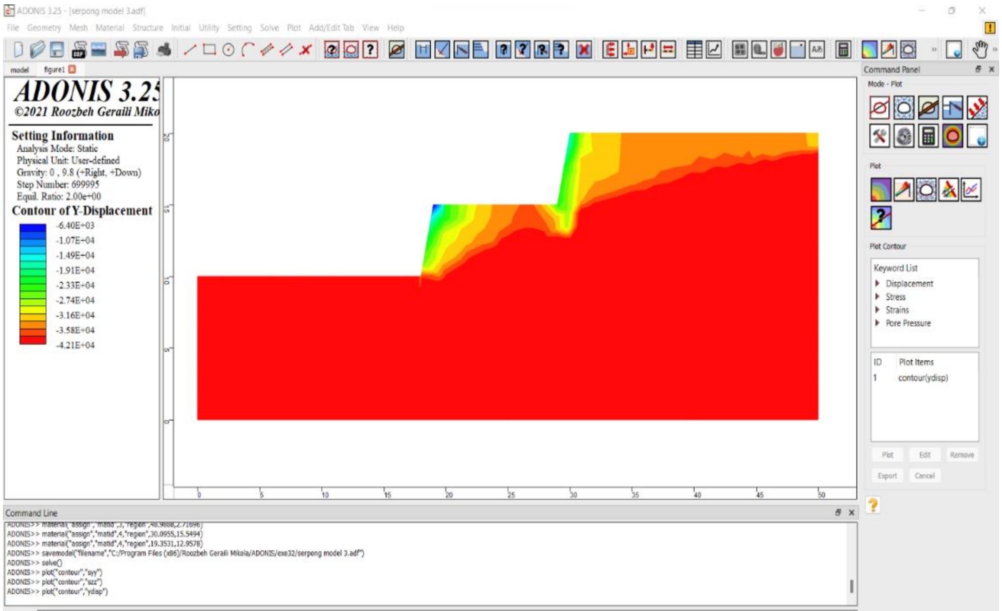

Figure 7 Displacement in the x-direction

The figure above shows the x-direction displacement of the retaining wall structure. The highest lateral movement (Sxmax) occurs at the top of the retaining wall at the bottom, with a displacement of 4mm.

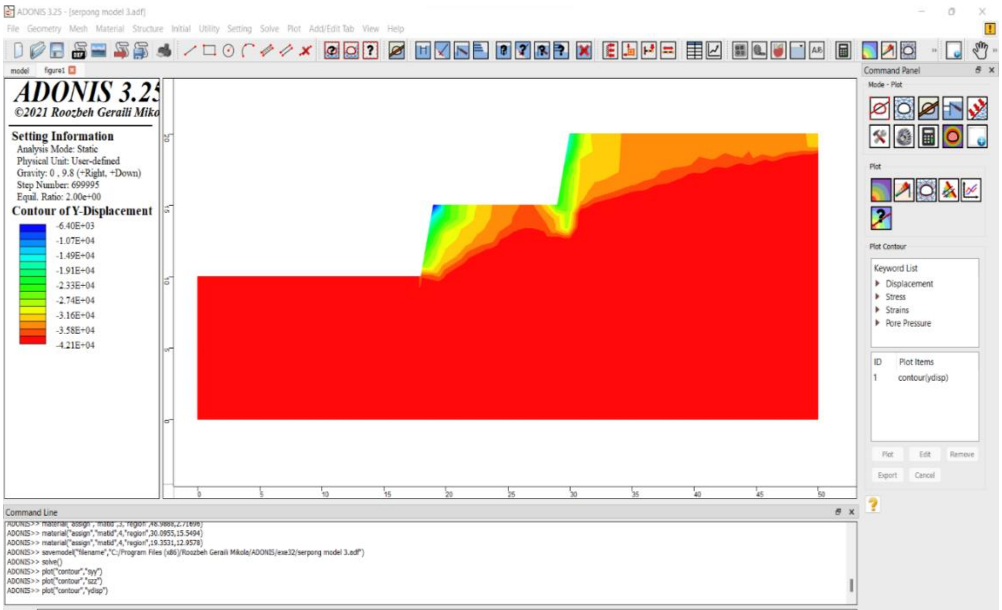

Figure 8 y-direction displacement

The figure above shows the y-direction displacement of the retaining wall structure. The highest vertical movement (settlement) (Symax) occurs at the top of the retaining wall at the bottom, with a movement of 0.4 mm.

Design of bored pile foundation retaining wall

Table 5 Dimensions of bored pile foundation planning

No.	Planning data	Notation	Value	Unit
1	On. bored pile foundation	DM/B	0,50	m
2	Bored pile foundation depth	D	9,0	m
3	Pile group length	L	25	m
4	Concrete quality	-	20,75	MPa
5	Steel quality	-	300	MPa

Source: Calculation results

Bored pile end resistance

Nominal end resistance is calculated by the formula:

Pb = w x Ab x qc

With:

w = reduction factor of the nominal end resistance of the pile,

Ab = area of the bottom end of the pile (m2),

qc = static cone penetration resistance which is the average value calculated from 8.D above the pile base to 4.D below the pile base (kN/m2),

Pile diameter, D = 0.50 m

The cross-sectional area of the pile,

$Ab = p / 4 * D^2 = 0.1963\ m^2$

Average static cone penetration resistance from 8.D above base to. 4.D under the base of the pile,

$qc = 70\ kg/cm2 \rightarrow qc = 7000\ kN/m^2$

The reduction factor for the nominal end resistance of the pile,

w = 0,50

Nominal pile end resistance:

Pb = w * Ab * qc = 687,22 kN

permit capacity= $687{,}22/2 = 343{,}66\ \mathrm{kN}$

Nominal frictional resistance according to Skempton is calculated by the formula:

$Ps = S\ [A_s * q_f]$

A_f = Surface area of the pile wall segment (m^2)

A_s = $p*D*L_1$

q_f = average static cone friction resistance (kN/m).

$P_s = S\ [A_s * q_f] = 276{,}85$

The axial resistance of the piles

Nominal pile resistance,

Pn = Pb + Ps = 964,08 kN

Strength reduction factor, f = 0.60

The axial resistance of the pile, f * Pn = 578.45 kN

Pile lateral resistance

The lateral resistance of the pile (H) for the long pile category can be calculated by the equation:

H = yo*kh*D/ [2*b*(e*b + 1)]

With,

b = $[k_h*D/(4*E_c*I_c)]0.25$

D = Diameter tiang pancang (m), D = 0,50 m

L = panjang tiang pancang (m), L = 8,00 m

kh = modulus tanah dasar horisontal (kN/m3), $k_h = 26000\ kN/m^3$

Ec = modulus elastis tiang (kN/m^2),

$Ec = 4700 * \sqrt{fc'} * 103 = 23500000\ kN/m^2$

Ic = momen inersia penampang (m^4)
Ic = p /64*D^4 = 0,003068 m^4
e = The distance of the lateral load to the ground (m), e = 0,20 m
y_o = maximum pile deflection (m), y_o = 0,006 m
b = pile deflection coefficient,
b = [kh*D/(4*Ec*Ic)]0,25
b = 0,46077764 m
b * *L* = 3,69 > 2,5, then including the long pole (OK)

Nominal lateral resistance of piles,
H = yo*kh*D/[2*b*(e*b+1)] **= 77,50 kN**
Shear strength reduction factor, f = 0,75
The lateral resistance of the pile, f * Hn = 58.02 kN
Calculation of Foundation Strength
Pilecap Material Data
Concrete compressive strength, fc' = 21 MPa
Yield strength of deformed reinforcing steel (□ > 12 mm), fy = 390 MPa
Yield strength of plain reinforcing steel (□ ≤ 12 mm), fy = 240 MPa
Weight of reinforced concrete, wc = 24 kN/m3
Foundation Dimension Data
Column width x direction, bx = 0.60 m
The column width in the y direction, by = 3.00 m
The distance between the edge piles and the outside of the concrete, a = 0.20 m
Pilecap thickness, h = 0.30 m
The thickness of the soil above the pile cap, z = 0.00 m
The unit weight of the soil above the pile cap, ws = 18.0 kN/m3
Foundation Load Data
Column axial force due to factored load, Puk = 252.10 kN
The x direction moment due to factored load, Mux = 139.52 kNm
The y direction moment due to factored load, Muy = 0.00 kNm
The x direction lateral force due to factored load, Hux = 95.10 kN
The y direction lateral force due to factored load, Huy = 0.00 kN
The axial resistance of the pile, Pn = 578.45 kN
The lateral resistance of the pile, Hn = 58.10 kN
Axial force on the pile
Soil weight above pilecap,
Ws = L_x * L_y * z * w_s = 0,00 kN
Berat pilecap, W_c = L_x * L_y * h * w_c = 17,28 kN
Total factored axial force, Pu = Puk + 1.2*Ws + 1.2*Wc = 272.84 kN
Max arm of pile x thd direction. center, xmax = 0.50 m
The maximum arm of the pile in the y direction to the center, ymax = 1.05 m
The minimum arm of the pile in the x thd direction. center, xmin = 0.30 m
Minimum arm pile in y direction to. center, ymin = 0.30 m
The maximum and minimum axial forces on the piles,

pu_{max} = Pu/n + Mu_x* x_{max} /Sx^2 + Mu_y* y_{max} / Sy^2 = 551,88 kN
pu_{min} = Pu/n + Mu_x* x_{min}/Sx2 + Mu_y* y_{min} / Sy2 = 440,26 kN

Condition: $pu_{max} \leq f * Pn$
551,88 < 578,45 → AMAN (OK)
Lateral force on the pile
The x-directed lateral force on the pile,
hux = Hux/n = 47.55 kN
The lateral force in the y direction on the pile,
huy = Huy/n = 0.00 kN

Two-way combined lateral force,
humax = √ (hux2 + huy2) = 47.55 kN
Terms: humax ≤ f * Hn
47.55 < 58.10 → SAFE (OK)
Pilecap reinforcement
1-way shear strength check
X-direction shear reinforcement
The distance from the center of the reinforcement to the outside of the concrete,
d' = 0.050 m
effective thickness of the pile cap,
d = h – d' = 0.250 m
bid distance. Critical of the outside
c_x = (Lx – bx – d) / 2 = -0,025 m
Weight of concrete, W1 = cx * Ly * h * wc = -0.540 kN
Soil weight, W2 = cx * Ly * z * ws = 0.00 kN
X direction shear force, Vux = 2 * pumax – W1 – W2 = 1104.292 kN
Width of the sliding plane for x-direction view,
b = Ly = 3000 mm
Effective pilecap thickness, d = 250 mm
Long side ratio to. short side of the column,
bc = bx/by = 0.2000
The shear strength of the pilecap in the x direction is taken from the smallest value of Vc obtained from the equation as follows:
Vc = [1+2/bc] *√fc'*b*d/6*10-3 = 6301.042 kN
Vc = [as*d/b+2] *√fc'*b*d/12*10-3 = 1288.849 kN
Vc = 1/3*√fc'*b*d*10-3 = 1145.644 kN
Taken, pilecap shear strength,
Vc = 1145.644 kN
Shear strength reduction factor, f = 0.75
Pilecap shear strength, f * Vc = 859.233 kN
Conditions that must be met, f * Vc ≥ Vux
859.233 < 1104.292 □ Not Safe (NG)
Y-direction shear reinforcement
The distance from the center of the reinforcement to the outside of the concrete,
d' = 0.05 m
effective thickness of the pile cap,
d = h – d' = 0.250 m
bid distance. Critical of the outside
cx = (Ly – by – d) / 2 = -0.125 m
Weight of concrete, W1 = cy * Lx * h * wc = -0.72 kN
Soil weight, W2 = cy* Lx * z * ws = 0.00 kN
y direction shear force,
Vux = 2 * pumax – W1 – W2 = 1104.292 kN
Width of the sliding plane for x-direction view,
b = Ly = 800 mm
Effective pilecap thickness, d = 250 mm
Long side ratio to. short side of the column,
bc = bx/by = 0.20
The shear strength of the pilecap in the x direction is taken from the smallest value of Vc obtained from the equation as follows:
Vc = [1+2/bc] *√fc'*b*d/6*10-3 = 1680.28 kN
Vc = [as*d/b+2] *√fc'*b*d/12*10-3 = 828.78 kN
Vc = 1/3*√fc'*b*d*10-3 = 305.51 kN
Taken, pilecap shear strength,

Vc = 305.51 kN
Shear strength reduction factor, f = 0.75
Pilecap shear strength, f * Vc = 229.13 kN
Conditions that must be met, f * Vc ≥ Vux
229.13 < 305.51 □ Not Safe (NG)
2 Way Slide Overview (Swipe Punch)
The distance from the center of the reinforcement to the outside of the concrete,
d' = 0.05 m
effective thickness of the pile cap,
d = h – d' = 0.25 m
Width of the punching shear in the x direction,
Bx = bx + d = 0.85 m
The width of the y-direction punch shear,
By = by + d = 3.25 m
Punch shear due to factored loads on the column
Puk = 252.10 kN
Punch shear area,
Ap = 2*(Bx + By) * d = 2.05 m2
Punch shear width,
bp = 2*(Bx+By) = 8.20 m
Long side ratio to. short side of the column,
bc = bx / by = 0.20
The puncture shear stress is taken as the smallest value of fp which is obtained from the following equation: fp = [1 + 2 / bc] * √ fc' / 6 = 8.401 Mpa
fp = [as * d / bp + 2] * √ fc’ / 12 = 1.113 Mpa
fp = 1 / 3 * √ fc' = 1.528 Mpa
The required punching shear stress,
fp = 1.113 MPa
Punch shear strength reduction factor,
f = 0.75
punch shear strength,
f * Vnp = f * Ap * fp * 103 = 1711.31 kN
Condition:
f * Vnp ≥ Puk
1711.31 > 252.10 □ SAFE (OK)
Flexural Reinforcement Details
tul. X-direction bending
Distance from the edge of the column to the outside of the pile cap,
cx = (Lx - bx) / 2 = 0.10 m
Pole distance to. column side,
ex = cx - a = -0.100 m
Concrete weight, W1 = cx * Ly * h * wc = 2.16 kN
Soil weight, W2 = cx * Ly * z * ws = 0.00 kN
The moment that happened on the pilecap,
Mux = 2*pumax*ex - W1*cx/2 - W2*cx/2
= -110.483 kNm
The reviewed pilecap width,
b = Ly = 3,000 mm
pile cap thickness,
h = 300mm
Rebar center distance to. concrete exterior,
d' = 50mm
The effective thickness of the plate,
d = h - d' = 250 mm

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concrete compressive strength,
fc' = 21 MPa
yield strength of reinforcing steel,
fy = 390 MPa
steel elastic modulus,
Ice = 2.00E+05 MPa
Teg distribution factor. Concrete
b1 = 0.85
rb = b1* 0.85 * fc'/ fy * 600 / (600 + fy)
rb = 0.023578089
Flexural strength reduction factor, f = 0.80
Rmax = 0.75*rb*fy*[1-½*0.75*rb*fy/(0.85*fc)]
Rmax = 5,564
Mn = Mux / f = -138.104 kNm
Rn = Mn * 106 / (b * d2) = -0.73655
Rn < Rmax □ (OK)
Required reinforcement ratio,
r = 0.85 * fc'/fy * [1-Ö{1– 2*Rn/(0.85*fc')}]
r = -0.0019
Minimum reinforcement ratio, rmin = 0.0025
The reinforcement ratio used, r = 0.0025
Required reinforcement area,
As = r * b * d = 1875.00 mm2
The diameter of the reinforcement used, D 13 mm
Required spacing,
s = p/4 * D2 * b/As = 454 mm
Maximum reinforcement spacing,
smax = 200mm
The spacing of the reinforcement used, □ s = 200 mm
Used reinforcement, D 13 - 200
used reinforcement area,
As = p/4*D2*b/s = 4252.93 mm2
tul. Direction-y bending
Distance from the edge of the column to the outside of the pile cap,
Cy = (Ly – by) / 2 = 0.00 m
Pole distance to. column side,
Ey = cy - a = -0.200 m
Weight of concrete, W1 = cy * Lx * h * wc = 0.00 kN
Soil weight, W2 = cy * Lx * z * ws = 0.00 kN
The moment that happened on the pilecap,
Muy = 2*pumax*ey - W1*cy/2 - W2*cy/2
= -220.75 kNm
The reviewed pilecap width,
b = Lx = 800 mm
pile cap thickness,
h = 300mm
Rebar center distance to. concrete exterior,
d' = 50mm
The effective thickness of the plate,
d = h - d' = 250 mm
concrete compressive strength,
fc' = 21 MPa
yield strength of reinforcing steel,
fy = 390 MPa

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steel elastic modulus,
Ice = 2.00E+05 MPa
Teg distribution factor. Concrete
b1 = 0.85
rb = b1* 0.85 * fc'/ fy * 600 / (600 + fy)
rb = 0.023578089
Flexural strength reduction factor, f = 0.80
Rmax = 0.75*rb*fy*[1-½*0.75*rb*fy/(0.85*fc)]
Rmax = 5.564
Mn = Mux / f = -275.94 kNm
Rn = Mn * 106 / (b * d2) = -5.52
Rn < Rmax □ (OK)
Required reinforcement ratio,
r = 0.85 * fc'/fy * [1-Ö{1– 2*Rn/(0.85*fc')}]
r = -0.0125
Minimum reinforcement ratio, rmin = 0.0025
The reinforcement ratio used, r = 0.0025
Required reinforcement area,
As = r * b * d = 500.00 mm2
The diameter of the reinforcement used, D 13 mm
Required spacing,
s = p/4 * D2 * b/As = 212 mm
Maximum reinforcement spacing,
smax = 200mm
The spacing of the reinforcement used, □ s = 200 mm
Used reinforcement, D 13 - 200
used reinforcement area,
As = p/4*D2*b/s = 530.93 mm2
Shrinkage Reinforcement
Minimum shrinkage reinforcement ratio,
rmin = 0.0014
The area of the x-direction shrinkage reinforcement,
Asx = rsmin* b * d = 1050 mm2
The area of shrinkage reinforcement in the y direction,
Asy = rsmin * b * d = 280 mm2
The diameter of the reinforcement used, Æ 10 mm
Distance of reinforcement in the x direction,
sx = w/4 * Æ2 * b / Asx = 224 mm
Distance of maximum shrinkage reinforcement in the x direction, sxmax = 200 mm
Distance of shrinkage reinforcement in the x direction used, sx = 200 mm
Shrinkage reinforcement spacing in the y direction, sy = p / 4 * Æ2 * b / Asy = 224 mm
Maximum shrinkage reinforcement distance in the y direction, symax = 200 mm
Distance of shrinkage reinforcement in the y direction used, sy = 200 mm
Shrinkage reinforcement used in the x direction, Æ 10 - 200
Shrinkage reinforcement is used in the y direction, Æ 10 - 200
Bored Piles Reinforcement
To determine the need for bored pile foundation reinforcement, then the reinforcement requirement analysis is divided into several analyses. The analysis is in the form of calculations regarding the main reinforcement requirements and shear reinforcement requirements.
Main reinforcement
Effective foundation thickness:
Foundation diameter = 500 mm

d^'=Concrete Cover+0.50 ×D ×Ø
d^'=50+0.50 ×16 ×10=68 mm
d=Diameter of foundation-d^'
d=500-68.00=432 mm
Foundation cross-sectional area:
Foundation diameter = 500 mm
Gross cross-sectional area of foundation:
A_(g)=1/4×3,14×D^2
A_(g)=1/4×3.14×500^2=196,250 mm^2
The ρmin limit according to SNI 03-2847:2002 Article 9.12 is 0.0020, then it is obtained:

$$A_{s\ needed} = \rho \frac{\pi d^2}{4}$$

$$A_{sneeded} = 0{,}0020 \times \frac{3{,}14 \times 432^2}{4} = 392{,}50\ \mathrm{mm}^2$$

Main reinforcement requirements:
Main reinforcement diameter
= 16 mm

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

$$A_{s\ \mathrm{reinforcement}} = 200{,}96\ \mathrm{mm}^2$$

amount of reinforcement needed:

$$n = \frac{A_{s\ \mathrm{needed}}}{A_{s\ \mathrm{reinforcemenr}}} = \frac{392{,}50}{200{,}96} = 1{,}96, 6\ \text{bars are taken}$$

So, the amount of main reinforcement for the bored pile foundation needed is 6D16. The total cross-sectional area of the longitudinal reinforcement:

$$A_{st} = 7 \times \frac{1}{4} \times 3{,}14 \times 16^2 = 1.205{,}76\ \mathrm{mm}^2$$

Minimum axial load strength:

$$\phi\ Pn\ (max) = 0{,}85\ \phi\ \left(0{,}85\ fc'\left(A_g - A_{st}\right) + (fy \times A_{st})\right)$$

$$\phi\ Pn\ (max) = 0{,}85 \times 0{,}70\ (0{,}85 \times 20{,}75 \times (196.250 - 1.205{,}76) + (300 \times 1.205{,}76))$$

$$\phi\ Pn\ (max) = 343.922{,}76\ N = 343{,}92\ kN > Pu$$

Spiral shear reinforcement
Noun shear strengthl:

$$V_n = \frac{V_u}{\emptyset}$$

$$V_n = \frac{47{,}55}{0{,}75} = 63{,}40 kN$$

Concrete shear force:

$$V_c = 1/6 \left(1 + \frac{P_u}{14\ A_g}\right) \sqrt{fc'} b_w d$$

$$V_c = 1/6 \times \left(1 + \frac{551{,}88}{14 \times 196.250}\right) \times \sqrt{20{,}75 \times 500 \times 432}$$

$$V_c = 164.020{,}744\ N$$

$$V_c = 164{,}02\ kN$$

because the value of ϕVn < ϕVc, the bored pile foundation requires shear reinforcement. The diameter of the spiral reinforcement used is Æ10. With the area of reinforcement used = 1/4×3.14×10^2=200.96 mm^2, the distance between s = p/4 * D2 * b/As = 200 mm is used. Thus, the spiral shear reinforcement used is Ø10-200.

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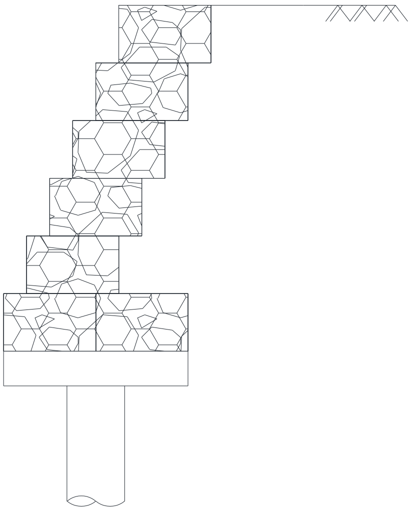

Figure 9 Retaining wall plans

Pile deflection check by p-y method

By using the L-pile ver. 4, the bored-pile response to lateral forces with the p-y method is shown in the pictures below.

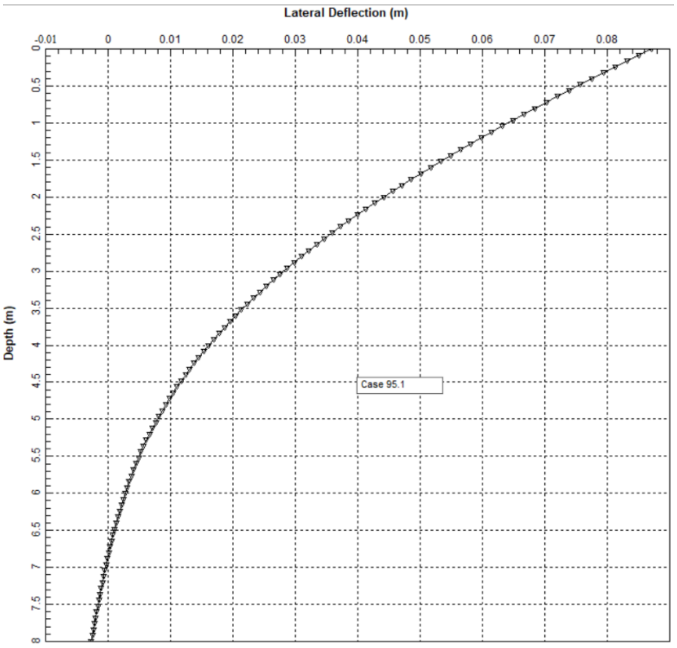

Figure 10 Lateral deflection of bored-pile vs depth

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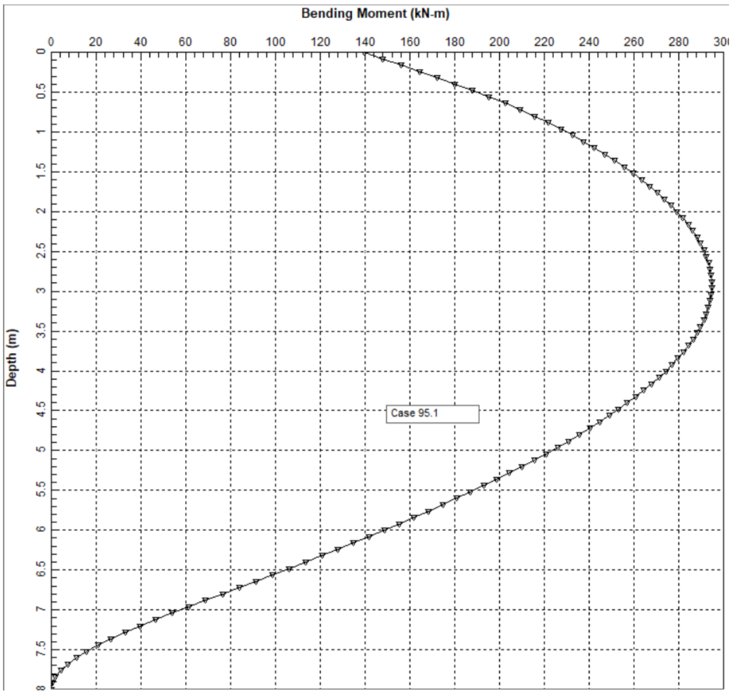

Figure 11 Distribution of actual moment on bored-pile vs depth

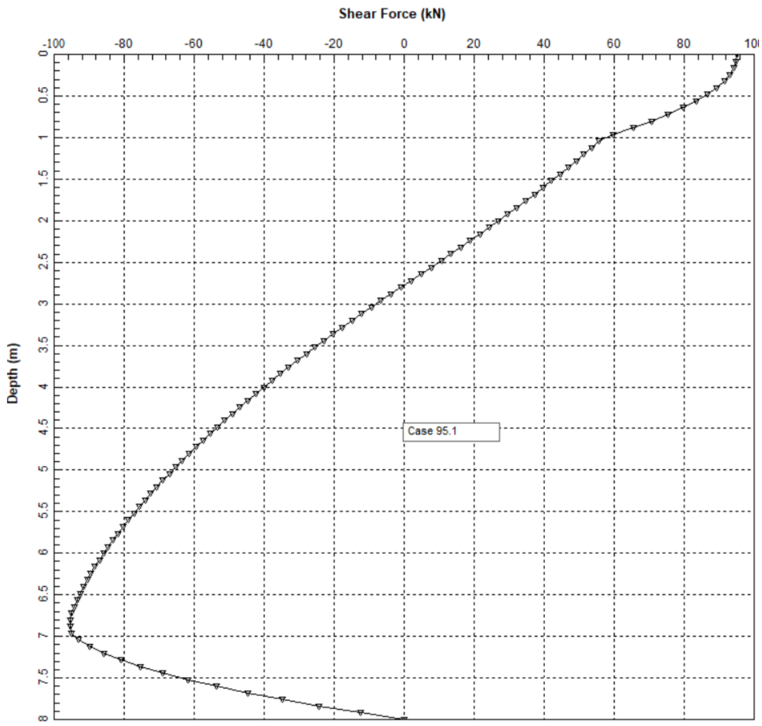

Figure 12 Distribution of shear forces on bored-pile vs depth

Based on Figure 9 above, it is shown that the maximum lateral movement of the pile (Smax) is at the top of the pile, which is 8.5 cm. Based on Figure 10, the maximum moment (Mmax) occurs at a depth of 3.0 meters, with an Mmax value of 295 kN.m. Based on Figure 11, the maximum shear force (Fmax) occurs at the top of the pile, with an Fmax value of 95.0 KN, while the minimum shear force (Fmin) occurs at a depth of 7.0 meters, with an Fmin value of -95.0 kN. Entire analysis is attached.

CONCLUSION

Based on the results of the study above, it can be concluded. The dimensions of the cantilever retaining wall construction are 4.5 meters high, the foot plate width is 0.8 meters, the foot plate height is 0.50 meters, and the top thickness of the retaining wall is 0.50 meters. The load acting on the retaining wall is 20 kN/m2. The active earth pressure load due to the earthquake based on the Mononobe-Okabe method, was 143.58 kN. Stability against shear is obtained at (Fgs) = 0.33 < SF = 1.5 and for stability against overturning (Fgl) = 0.45 < SF = 1.5, the stability to shear and stability to roll is not safe. The stability of the bearing capacity of the soil obtained a safe ultimate capacity (qs) = 252.10 kN/m2 > V = 120.0 kN/m2, so that the collapse of the bearing capacity of the soil is declared safe. Bored pile foundation is designed with a depth of 8.0 meters and a diameter of 0.50 meters, with an axial clearance capacity of 578.45 kN, and a lateral capacity of 58.02 kN. Bored pile main reinforcement 6-D13, and spiral shear reinforcement 10-200, are installed every 3.0 meters. The pile cap is planned to have a plate thickness (h) of 30 cm, and a tread width (B) of 80 cm, with x-direction reinforcement D13-200, y-direction reinforcement D13-200, and shrinkage reinforcement 10-200. The maximum lateral movement of the pile (Smax) is at the top of the pile, which is 8.5 cm. The maximum moment (Mmax) occurs at a depth of 3.0 meters, with a Mmax value of 295 kN.m. The maximum shear force (Fmax) occurs at the top of the pile, with an Fmax value of 95.0 KN, while the minimum shear force (Fmin) occurs at a depth of 7.0 meters, with an Fmin value of -95.0 kN.

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