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Planning of the foundation a three-story building constructed on potential liquefaction area in Air Tawar Estuary of Padang City

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Abstract. The earthquake has become one of the most lethal problems in Indonesia, especially across the ring of the fire zone. On September 30th, 2009, an earthquake with a magnitude of 7.6 SR occurred, triggering tsunami and liquefaction. The liquefaction potential and tsunami forces need to be mitigated when an earthquake happens to reduce the risk. This study designs the building foundation that can endure the loads from an earthquake, tsunami, and liquefaction in the Air Tawar estuary of Padang City. The soil profile and liquefaction potential can be identified with CPT (Cone Penetration Test) data. After identifying the liquefaction potential, the upper structure was designed to consider the earthquake load and tsunami waves according to FEMA P-464 (2012). Afterward, design the foundation dimensions based on the ultimate load from the upper structure, wherein the amount of the settlement should be smaller than the tolerable soil settlement. The method used in the paper is taking the CPT data in the Air Tawar Estuary. After analyzing the liquefaction potential, making an upper structure design with ETABS 2016, recapitulate the maximum joint reaction to design the foundation that can withstand the ultimate load, and calculate the amount of soil settlement. Thus the design of the upper structure and lower structure take into account liquefaction. The amount of soil settlement obtained in the building design is still within the tolerance range of 9.79 mm, where the maximum limit of reduction is 32m.

1 Introduction

The building is known as one of the fundamental human needs. These days, the building can become the most dangerous place during natural disasters, especially earthquakes and tsunamis. Cyclic load from an earthquake can also trigger liquefaction, especially in the area around the coast and estuary river. The results of research conducted by Yuliet et al. at Nurul Haq shelter that is located close to the coastline, with a distance of 0.37 km towards the coastline of Padang, has a potential of liquefaction between the depths of 11.55 m and 33.55 m [1].

Putra et al. researched liquefaction potential at GOR Haji Agus Salim and Lapai, Padang City. On research conducted an analysis using CPT method, which focused on the correlation between cone penetration against cyclic stress ratio according to Seed and Idris chart which described the soil boundary whether the liquefaction occurs or not. Compared to a predetermined safety factor of 1.50. It is concluded that the research area has a considerable liquefaction potential [2]. Based on previous research in Padang city, in planning the foundation for high-rise buildings and considering the tsunami load, it is also necessary to consider soil conditions in areas with liquefaction potential.

2 Study Literature

2.1 Liquefaction Potential Analysis

Liquefaction is a natural disaster where the strength and stiffness of the soil are reduced due to earthquakes or other ground movements. It is a process of changing the nature of the soil from solid to liquid, caused by cyclic loads when an earthquake occurs so that the pore water pressure increases close to or exceeds the vertical stress. In this case, the soil undergoing liquefaction is sandy soil or contains much sand, which means that the soil is non-cohesive and saturated. Liquefaction occurs in loose sandy soil (not dense) and saturated with water. As the water pressure increases caused by earthquake shocks, the effective stress decreases [3].

During an earthquake, it is usually followed by a series of cyclic loads and sometimes a tsunami. One of the impacts of soil shaking and soil distribution in specific geological environments is liquefaction. This process can cause buildings to crack, damage, and even collapse; damage to buildings due to liquefaction is called soil failure. One way to evaluate liquefaction potential is to use CPT data, which in this equation compares the CRR value to the CSR value, which can be explained in equation (1) [4]:

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$$FS = \frac{CRR}{CSR} \quad (1)$$

FS < 1.50 Liquefaction not occurs
FS > 1.50 Liquefaction occurs

Before evaluating the liquefaction potential, the profile and soil properties of Air Tawar Estuary should be determined first, which will then be used to evaluate the liquefaction potential. In classifying soil, the Robertson and Wride methods are used [5], it can be seen in Fig. 1.

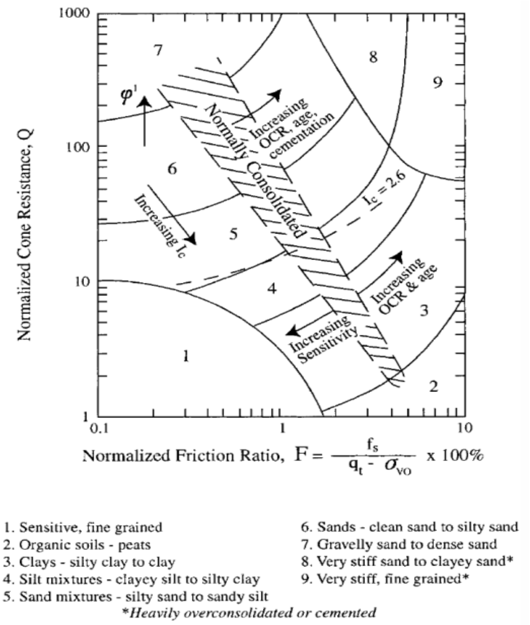

Fig 1. Soil Classification using Robertson and Wride methods [5].

To obtain other soil properties, we can use the correlation Table below [6]:

Table 1. Soil strength based on CPT data [6]

Soil classification		Estimated value qc (MPa)
Very Soft	Cu = 0-12 kPa	<0.2
Soft	Cu = 12-25 kPa	0.2-0.4
Firm	Cu = 25-50 kPa	0.4-0.9
Stiff	Cu = 50-100 kPa	0.9-2.0
Very stiff	Cu = 100-200 kPa	2.0-4.2
Hard	Cu > 200 kPa	> 4.0

Table 2. Preliminary sand strength from CPT [6]

Relative Compaction D_r (%)		q_c (MPa)	Internal Friction, ϕ (°)
Very loose	Dr < 15	<2.5	<30
Loose	Dr = 15-35	2.5 – 5.0	30 – 45
Med dense	Dr = 35-65	5.0 – 10.0	35 – 40
Dense	Dr = 65-85	10.0 – 20.0	40 – 45
Very dense	Dr >85	>20.0	>45

.

Table 3. A representative range of dry unit weight [6]

Type	Soil description	Unit weight range (kN/m^3)	
		Dry	Saturated
Cohesion less Compacted broken rock	Soft sedimentary	12	18
	Hard sedimentary	14	19
	Metamorphic Igneous	18	20
Cohesion less; sands and gravel	Very loose	14	17
	Loose	15	18
	Medium dense	17	20
	Dense	19	21
	Very dense	21	22
Cohesion less sands	Loose-Uniformly graded	14	17
	Loose-Well graded	16	19
	Dense-Uniformly graded	18	20
	Dense-Well graded	19	21
Cohesive	Soft-organic	8	14
	Soft-non organic	12	16
	Stiff	16	18
	Hard	18	20

The Coefficient Stress Ratio (CSR) value (Seed and Idris, 1971) [5],

$$CSR = (\tau_{av}/\ \sigma'_{vo}) = 0.65\ (a_{max}/g)\ (\sigma_{vo}/\ \sigma'_{vo})r_d \quad (2)$$

Where the shear stress reduction coefficient (r_d) is, [7]:

$$r_d = \frac{(1.000 - 0.04113z^{0.5} + 0.04052z + 0.001753z^{1.5})}{(1.000 - 0.04177z^{0.5} + 0.05729z - 0.006205z^{1.5} + 0.001210z^2)} \quad (3)$$

To determine the Cyclic Resistance Ratio (CRR) value, use equation (4) and equation (5) [7],

If $(q_{c1N})_{cs} < 211$ (4)

So, CRR= exp $[((q_{c1N})_{cs} / 540) + ((q_{c1N})_{cs} / 67)^2 - ((q_{c1N})_{cs}\ 80)^3 + ((q_{c1N})_{cs} / 114)^4 - 3]$

If (qc1N)cs > 211 (5)

So, CRR7.5= 2

2.2 Earthquake

According to the USGS earthquake, the term is used to describe the sudden slip of a fault. The resulting ground is shaking and emitting seismic energy caused by the slip, volcanic or magmatic activity, or other sudden changes in the earth's stress. The 2017 Indonesia Earthquake Source and Hazard Map were launched by the Ministry of Public Works and Public Housing (PUPR), This is done to increase preparation for the potential for earthquakes in the future to minimize losses. The 2017 earthquake map can be seen in Fig. 2

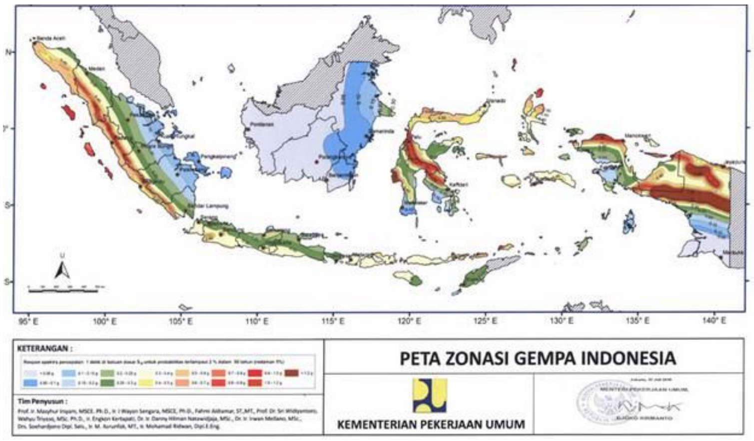

Fig 2. Earthquake and hazard maps 2017 [8]

2.3 Tsunami

Tsunami comes from Japanese, composed of the words *tsu* which means harbor, and *nami* which means wave. Tsunamis are ocean waves that are formed due to impulsive disturbances in the ocean. The disturbance is formed due to a sudden change in seabed shape, either horizontal or vertical direction. These changes are caused by three primary sources: volcanic eruptions, tectonic earthquakes, or landslides on the seabed.

The tsunami loads are calculated based on FEMA P-464 2012 [9], as follows:

1. Hydrostatic loads
   The loads are applied to the center of the column as high as 1/3 of the maximum water height in the direction of the tsunami wave.
2. Buoyancy
   The buoyant force is concentrated at the center of mass of the floor plate submerged by the tsunami
3. Hydrodynamic loads
   The hydrodynamic loads located in the column as high as 1/2 of the maximum water height, in the direction of the tsunami wave
4. Impulsive forces
   The impulse forces are distributed on the walls of the structure at the height of the tsunami wave in the direction of the tsunami.
5. Debris impact forces
   The impact force from waterborne debris (e.g., floating driftwood, lumber, shipping containers, automobiles, boats) can cause significant building damage.
6. Damming of accumulated waterborne debris
   The force due to debris is uniformly applied to the structural element with a minimum width of 40 ft (12 m)
7. Uplift forces on elevated floors
   The uplift forces act evenly on the upper floors affected by the tsunami
8. Additional Retained Water Loading on Elevated Floors

2.4 Pile Foundation

The method used in the calculation of the pile foundation is a static analysis method. The static analysis method can be categorized as an analytical method that uses the compressibility and strength properties of the soil to determine the pile performance and capacity. Static pile capacity from the amount of soil/rock resistance along the sides of the pile and at the tip of the pile can be estimated from geotechnical engineering analysis using the following data:

a. Laboratory test data to determine the soil shear strength and rock around the pile
b. Standard Penetration Test /SPT data
c. Cone Penetration Test /CPT

The calculation of the foundation used the direct method proposed by Meyerhof (1956). The equation can be seen in equation (6) [10].

$$P_{allow} = \frac{q_c \times A_p}{3} + \frac{JHP \times K_{ll}}{5} \quad (6)$$

3 Analysis and Discussion

3.1 Liquefaction Potential Analysis

The soil classification is obtained by using the Robertson and Wride method, as shown in Fig. 3.

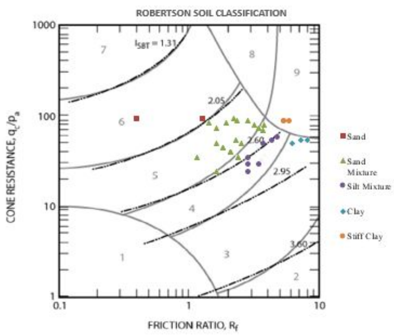

Fig 3 Soil classification in Air Tawar estuary

Analysis of liquefaction potential using the CPT method, while the CPT graph is as follows:

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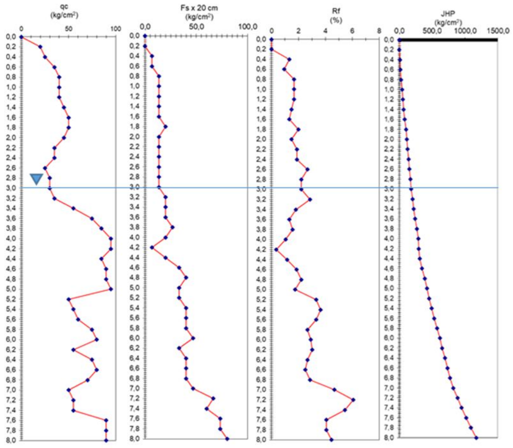


Fig 4. CPT graph in Air Tawar estuary

Table 4. Recapitulation of CSR, CRR, and FS values at each depth

Depth (m)	Soil type	CRR	CSR	FS	Description
0.0	Sand	Not evaluated			
0.2					
0.4	Sand mixture				
0.6					
0.8					
1.0					
1.2					
1.4					
1.6					
1.8					
2.0					
2.2					
2.4					
2.6	Silt mixture				
2.8					
3.0					
3.2		0.087	0.333	0.521	L
3.4	Sand mixture	0.114	0.345	0.565	L
3.6		0.156	0.356	0.622	L
3.8		0.179	0.367	0.794	L
4.0	Sand	0.206	0.376	0.713	L
4.2		0.200	0.385	0.520	L
4.4	Sand mixture	0.166	0.393	0.566	L
4.6		0.175	0.400	0.830	L
4.8		0.171	0.407	1.017	NL
5.0		0.180	0.413	0.792	L
5.2	Silt mixture	0.110	0.420	0.942	L
5.4		0.118	0.426	1.412	NL
5.6		0.126	0.432	1.269	NL
5.8	Sand mixture	0.127	0.437	0.762	L
6.0		0.134	0.441	0.999	L
6.2	Silt mixture	0.107	0.446	0.706	L
6.4	Sand mixture	0.122	0.450	0.696	L
6.6		0.129	0.453	0.682	L
6.8		0.111	0.456	0.662	L
7.0	Clay	0.091	0.460	1.405	NL
7.2		0.097	0.464	4.308	NL
7.4		0.095	0.468	4.037	NL
7.6	Stiff clay	0.154	0.471	4.249	NL
7.8		0.150	0.473	4.225	NL
8.0		0.147	0.476	4.203	NL

Note: L = Liquefaction, NL = Not Liquefaction

By using the correlation table in Table 1, Table 2, and Table 3, the soil profile in the Air Tawar Estuary is obtained, which is described in Table 4. After obtaining the properties and soil profile, CSR, CRR, and Safety of Factor are sought at each depth to evaluate the liquefaction potential, as shown in Table 4. From Table 45, it can be seen that the soil in the Air Tawar Estuary area has the potential for liquefaction where all Safety of Factor values are several depths smaller than 1.

3.2 Upper Structure Design

3.2.1 Building structure data

In conducting a structural analysis, the parameters used in the technical data of the building and the building design conditions are as follows:

Address	: Air Tawar Estuary, Padang	
Structure Type	: Reinforced Concrete	
Number of Floors	: 3	
Building Height	: 1st floor	(8 m)
	2nd, 3rd Floor	(4 m)
Building Area	: 320 m^2	
Building Function	: Shelter	
Floor Plate Thickness	: 14 cm	
Beam Dimensions	: 600 mm x 300 mm	
Column Dimensions:	Column K_1 (800 mm x 800 mm)	
	Column K_2 (600 mm x 600 mm)	

3.2.2Structural modelling

Fig. 5 shows a 3D model of the building designed using ETABS 2016, and the floor slabs are modeled as slabs, beams, and columns are modeled as frames.

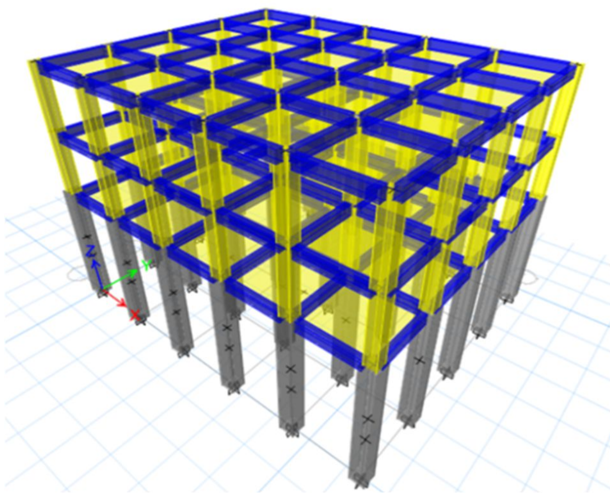


Fig. 5. Three-story building structure modelling

3.2.3 Load analysis

Load analysis refers to : PPIUG 1983 [11] ; SNI 1727-2013 [12] ; SNI 1726-2019 [13]. The living load of refugees used is 250 kg/m^2 (FEMA P-464 2012) [9]. SPT value was 6.15, which means the soil in the Air Tawar can be categorized as soft soil (SE) [14], where the N-SPT value is smaller than 15. The response spectrum can be seen in Fig. 6.

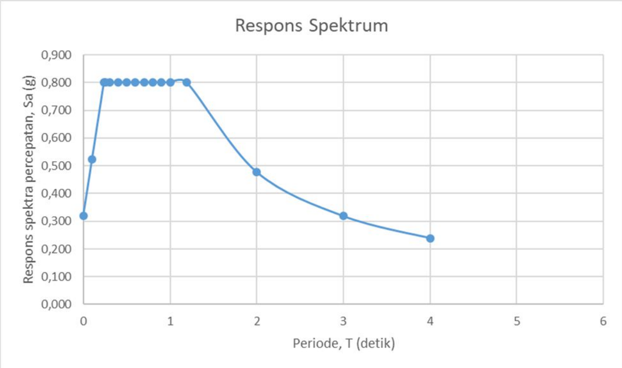


Fig. 6. Soil spectrum response

For tsunami loads recapitulation based on FEMA P-464, 2012 [10] can be seen in Table 6.

Table 6. Tsunami load recapitulation

No.	Tsunami Load	Load (kN)
1	Hydrostatic loads on the wall	236.40
	Hydrostatic loads on the front column	590.87
	Hydrostatic loads on a column	276.25
2	Floating force	10,860.06
3	1-floor hydrodynamic loads	101.00
	2-floor hydrodynamic loads	233.57
4	Impulsive force	151.51
5	Debris impact force	647.29
6	Damming of accumulated waterborne debris	5,908.72
7	Uplift forces on elevated floors	0
8	Additional retained water loading on elevated floors	39.93 kN/m^2

For the combination of loading using FEMA P-464 2012 [9] and SNI 1726:2012 [13], where the maximum reaction value can be seen in Table 7.

Table 7. Maximum reaction value

Force (kN)			Momen (kN)		
F_x	F_y	F_z	M_x	M_y	M_z
27.90	33.76	1,217.70	3,842.89	45.90	1.23

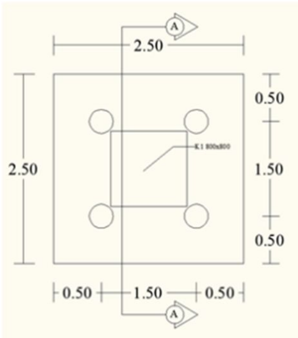
For the foundation calculation, only use the F_z value of 1,217.70 kN.

3.3 Foundation Design

The foundation design is carried out by trial and error using the direct cone method as described in equation (5). The foundation size design is obtained as follows:

a. Pile Cap (Fig. 7.):
- Thick (D) : 80 cm
- Length (P) : 200 cm
- Width (B) : 200 cm

b. Foundation (Fig. 8.):
- Number of Piles : 4
- Pile Height : 720 cm
- Pile Diameter : 30 cm
- Pile Material : Reinforced concrete



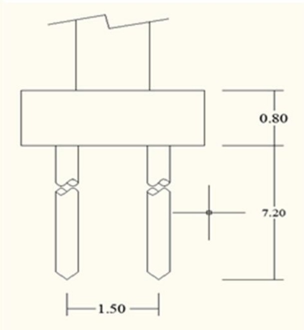


Fig. 7. Pile cap plan **Fig. 8.** A-A pile cap section

3.3.1 Foundation bearing capacity

In planning the bearing capacity of the foundation, only the combination of live load, dead load, and earthquake load is considered, which is the value of P_{max} borne by the foundation resulting from a combination of axial force and moment. At the same time, the value of the most significant force and moment obtained is 727.02 kN for axial force, 319.31 kN-m for a moment in the x-direction, and 147.75 kN-m in the y-direction. Then look for the P_{max} value with the following equation (7):

$$P_{max} = \frac{F_z}{n} + \frac{M_x.x_1}{n_x\,(2.x_1^2)} + \frac{M_y.y_1}{n_y\,(2.y_1^2)} \tag{7}$$

$$P_{max} = \frac{727.02}{4} + \frac{319.31\times0.75}{2\,(2\times0.75^2)} + \frac{147.75\times0.75}{2\,(2\times0.75^2)} = 337.42 \text{ kN}$$

The maximum load (P_{max}) value for the building is 337.42 for one pile. The value of the single pile-soil bearing capacity uses equation (7) where Cross-sectional area (A) = 0.07 m^2 and Circumference (K_{ll}) = 0.94 m as follows:

$$P_{allow} = \frac{9{,}000\times0.07}{3} + \frac{673.33\times0.94}{5} = 338.98 \text{ kN}$$

It was found that the single pile bearing capacity is 338.98 kN, while the bearing capacity of the pile group is 1355.91 kN, of which the single pile bearing capacity of 338.98 kN could withstand the P_{max} of the superstructure of 337.42 kN. It can be concluded that four piles with a diameter of 30 cm can withstand the ultimate load from the building. The design of the foundation and soil profile can be seen in Fig. 9

3.3.2Soil settlement

In the construction of buildings, the magnitude of soil settlement should be reviewed to reduce the impact on the building itself. The amount of tolerance for pile settlement according to Skempton and McDonald in 1956 can be seen in Table 7, while the magnitude of the settlement in the pile group for consolidation settlement and quick settlement can be seen in equation (8) and equation (9) [15]:

$$S_c = \frac{C_c.H_c}{1+e_o} log \frac{\sigma_o' + \Delta\sigma'}{\sigma_o'} \quad (8)$$

$$S_i = H \frac{1}{C'} log \frac{\sigma'_0 + \Delta\sigma'}{\sigma'_0} \quad (9)$$

Table 7. Pile group settlement tolerance [15]

Maximum settlement	
Sand	32 mm
Clay	45 mm

In equation (8), several soil properties have not been obtained. To find the value of C_c the Sanglerat method (1972) is used, which can be seen in equation (10) as follows [16]:

$$C_c = \frac{q_c - 4,2}{6q_c - 4,0} \quad (10)$$

After calculating the amount of soil settlement, the magnitude of the value of soil settlement for the equivalent layer under the foundation using the 4V:1H method can be seen in Table 8. The value of the pile group settlement is 9.79 mm, which meets the settlement limits in Table 7, which for the settlement tolerance is 32 mm.

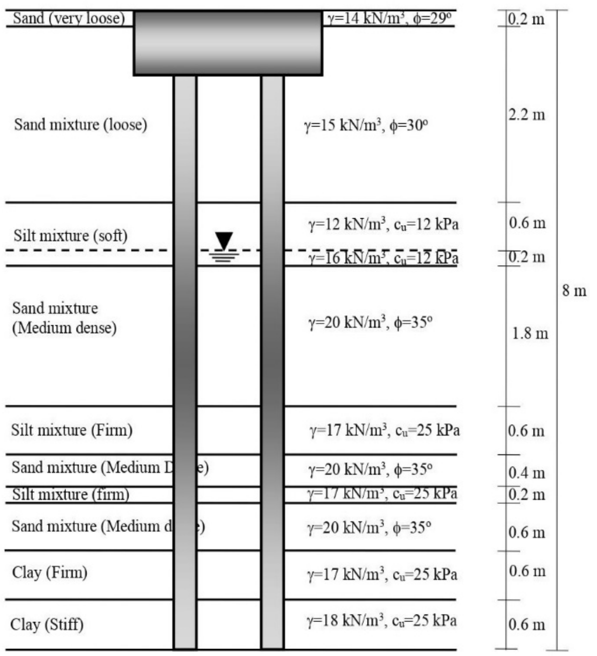


Fig. 9. Soil profile and pile design in Air Tawar Estuary

Table 8. The amount of soil settlement under the foundation is equivalent

Layer	Depth (m)	Settlement type	Total settlement (mm)
8	5.6 - 6.0	Quick settlement	1.1
9	6.0 - 6.2	Quick settlement	0.96
10	6.2 - 6.8	Quick settlement	1.21
11	6.8 - 7.4	Consolidation settlement	2.78
12	7.4 - 8.0	Consolidation settlement	3.74
The total settlement of the pile group			9.79

4 Conclusion

From the results of testing the CPT data, it was found that:

1. The soil profile in the Muara Air Tawar area is dominated by a mixture of sand consisting of 12 layers with the groundwater table at a depth of 3 m below the ground surface.
2. The analysis of the potential for liquefaction using CPT test data, it was found that the soil in the Muara Air Tawar area has the potential for liquefaction at a depth of 3 m - 4.6 m, 4.8 m - 5.2 m, and 5.6 m - 6.8 m with the lowest FS value of 0.52 at a depth of 4.2 m.
3. The planning design of a 3-story building in the Muara Air Tawar area with a height of the 1st floor as high as 8 m, the 2nd and third floors each 4 m, the beam dimensions are 600 x 300 mm, the column dimensions of the 1st floor are 800 x 800 mm and the dimensions of the 2nd and 3rd-floor columns are 600 x 600 mm, while the thickness of the floor slab is 140 mm thick.
4. The foundation design considers the liquefaction potential; the pile cap dimensions are 250 x 250 cm with a thickness of 80 cm, while the number of piles obtained is 4 per column point, with a length of 720 cm and a pile diameter of 30 cm.
5. The soil settlement that occurred was still in the safe category because the total settlement was 9.79 mm, where the maximum settlement limit on sandy soil is 32 mm.

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