Analysis of Liquefaction Potential at Public Facilities in Petobo Area Based on CPT and SWS Test Results

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ABSTRACT: The liquefaction event that occurred in Petobo Village, Palu City, Central Sulawesi was caused by an earthquake on September 28, 2018 with a magnitude of 7.4 and a depth of 10 km. This study aims to determine the potential for liquefaction and the bearing capacity of the soil at SDN Inpres Petobo based on the Swedish Weight Sounding (SWS) and Cone Penetration Test (CPT) tests. The test was conducted at SDN Inpres Petobo, Petobo Village, Palu City, Central Sulawesi. Determination of liquefaction potential and soil bearing capacity using SWS as primary data and CPT as secondary data and also supported by laboratory tests of sieve analysis, Atterberg limits, and direct shear. The sieve analysis test obtained SP classification of soil types at all points. The Atterberg limits test obtained PI values < 7, so that it entered the criteria for silty soil, low plasticity, and partially cohesive, and LL values < 35% which is a type of soil that has the potential for liquefaction. In direct shear testing, the cohesion value at point T1 is 0.225 kg/cm^2 with a shear angle of 13.534° and at point T2 the cohesion value is 0.19 kg/cm^2 with a shear angle value of 35.829°. Based on the results of the SWS test, at point T1 at a depth of 0 m to 5.5 m, liquefaction has the potential to occur as indicated by the FS value less than 1. From the results of the CPT calculation, it was found that the soil at point T1 with a depth of 0.20 m to 4.40 has the potential for liquefaction to occur. However, the liquefaction did not occur because of its location in the upper side of the Gumbasa irrigation canal and the groundwater table was deep. The calculation of bearing capacity for shallow foundations with SWS data, shear strength data and CPT data, obtain that at a depth of 1 m with dimensions of 1 m x 1 m foundation size is sufficient to support simple construction loads**.**

Keywords: liquefaction, sieve analysis, atterberg limits, direct shear test, swedish weight sounding (SWS), cone penetration test (CPT)

1 INTRODUCTION

Indonesia is a country located between the three main plates of the earth, namely Australia, Eurasia and the Pacific. This condition causes Indonesia frequently experience tectonic earthquakes and volcanic earthquakes. Earthquakes can occur due to collisions between the main plates. Earthquakes that occur in Indonesia vary, from small to large scale. In fact, most of the archipelago is very prone to earthquakes, one example is the area of Central Sulawesi Province.

The liquefaction event that occurred in Petobo Village, Palu City, Central Sulawesi, was caused by the Palu earthquake, September 28th, 2018 with a magnitude of 7.4 Mw and a depth of 10 km, BMKG (2018). The earthquake occurred due to land fault activity, namely the Palu Koro Fault. The Palu Koro Fault is a fault that extends approximately 240 km from the north (Palu City) to the south (Malili) to Bone Bay.

SD Inpres Petobo is a public facility located in the Petobo area and is still being used today. The location of this school is right in a liquefaction-prone area. So, it is necessary to countermeasures to reduce the impact of liquefaction that occurs.

This research aims to determine the potential of liquefaction from the results of the SWS and CPT tests and to determine the bearing capacity of the soil from SWS and CPT data at public facilities in the Petobo area, in this case SD Inpres Petobo.

2 LITERATURE REVIEW

2.1 *Earthquake*

An earthquake is an event that vibrates the earth due to the sudden release of energy in the earth which is marked by the breaking of rock layers in the earth's crust. The accumulation of energy that causes earthquakes results from the movement of tectonic plates. The energy produced is emitted in all directions in the form of earthquake waves so that its effects can be felt up to the earth's surface.

According to John Tri Hatmoko, an earthquake is defined as a sudden movement or a series of sudden movements of soil and rock that are transient and originate from a limited area that may spread in all directions because they are propagated by the existing medium (layers of the earth). An earthquake is defined as an original jolt that occurs in the earth, originating from within the earth which then propagates on the surface.

The two definitions above provide several main points, which basically means that in an earthquake there must be:
• Vibration occurs suddenly;
• There is or has a source;
• The presence of vibration / wave propagation.

2.2 *Liquefaction*

In the event of an earthquake, it is generally followed by a series of shaking and ground shaking as a result of the earthquake waves reaching the surface and sometimes causing a Tsunami. One of the causes of liquefaction is soil shaking and soil displacement in certain geological environments.

Liquefaction is a phenomenon where the strength of the soil layer is lost due to earthquake vibrations. Liquefaction occurs in loose sandy soil (not dense) and saturated with water, Towhata (2008). Liquefaction can lead to deformation of the soil surface. When liquefaction occurs, the sand layer turns into a liquid so that it is unable to support the load of the building inside or above it. A process of loss of soil shear strength due to an increase in soil pore water stress arising from cyclic mobility.
Liquefaction only occurs in saturated soils, so the effect is often only observed in areas close to water bodies such as rivers, lakes, and oceans. The effects caused by liquefaction can be in the form of large landslides or the occurrence of cracks in the soil parallel to the body of water.

When liquefaction occurs, the strength of the soil decreases and the ability of the soil to support the foundation of the building above it decreases. Liquefaction can also put great pressure on retaining walls which can cause retaining walls to tilt or shift.

2.3 *Sieve Analysis*

The properties of granular soils are strongly influenced by the grain size. One method for classifying soil types is based on the distribution of grain size (gradation). Sieve analysis is a mechanical means of analyzing the grain size distribution of coarse-grained soils (ie. the grains retained on the No. 200 sieve). The amount of soil retained in each sieve is measured, and the cumulative percentage of soil passing through each is determined, and the formula used is:

$$\% \text{ Finer} = \frac{\text{Wpass}}{\text{Wtotal}} \times 100\ \%. \quad (1)$$

Types of soil that are susceptible to liquefaction are loose sandy to medium saturated soils. Tsuchida (1970) summarized the results of sieve analysis on a number of soils known to have liquefied and which were not liquefied during an earthquake.

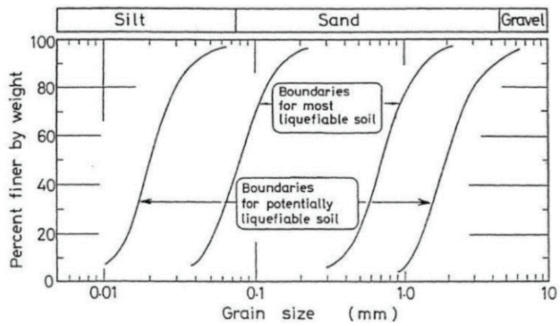

Fig. 1. Grain Size Distribution Curve of Soil Susceptible to Liquefaction, Tsuchida (1970).

2.4 *Direct Shear*

Soil shear strength is the resistance force exerted by soil grains against pressure or pull, Hardiyatmo (1992). If the soil is subjected to loading it will be restrained by:
- Soil cohesion depends on the type of soil and its density, but does not depend on the normal stresses acting on the shear plane.

- Friction between soil grains whose magnitude is directly proportional to the normal stress in the shear plane.

Coulomb (1776) define τ_f as :

$$\tau_f = c + \sigma . \tan \varphi \quad (2)$$

where τ is shear strength (kN/m^2), c is cohesion, σ is normal stress (kN/m^2) and φ is angle of friction (°).

The value of the internal shear angle (φ) and cohesion (c) obtained from this test can be used to determine the type of soil and the density of the soil sample being tested.

2.5 *Atterberg Limit*

In the early 1900s, a Swedish scientist named Atterberg developed a method to describe the consistency of fine-grained soils at varying water content. If the water content is very high, the soil and water mixture will become very soft like liquid.

When clay is mixed with excess water, the soil can flow like a semiliquid. If the soil is gradually drained, it will behave like a plastic material, semi-solid, or solid, depending on its moisture content. The moisture content in percent, at which the soil changes from a liquid to a plastic state is defined as the liquid limit (LL). Similarly, the moisture content, in percent, at which the soil changes from plastic to semisolid and from semisolid to solid state is defined as plastic limit (PL) and shrinkage limit (SL), respectively. These limits are referred to as the Atterberg Limits, Das (2011).

The Plasticity Index (IP) is the difference between the liquid limit and the plastic limit of a soil. Plastic Index is calculated based on the formula:

$$IP = LL - PL \quad (3)$$

where IP is plastic index, LL is liquid limit and PL is plastic limit.

2.6 *Swedish Weight Sounding (SWS)*

Swedish Weight Sounding (SWS) was first introduced by the Swedish Geotechnical Committee of State Railways. SWS is a tool that has often been used for geotechnical investigations caused by earthquakes. This test can be used for various purposes such as soil profiling, determination of bearing capacity of shallow and deep foundations, determination of engineering characteristics of soils such as shear strength or compressibility, evaluation of liquefaction resistance or soil liquefaction as a transformation of granular material from a solid to a liquid state as a consequence of increasing pressure, pore water and reduced effective stress, Marcuson (1978).

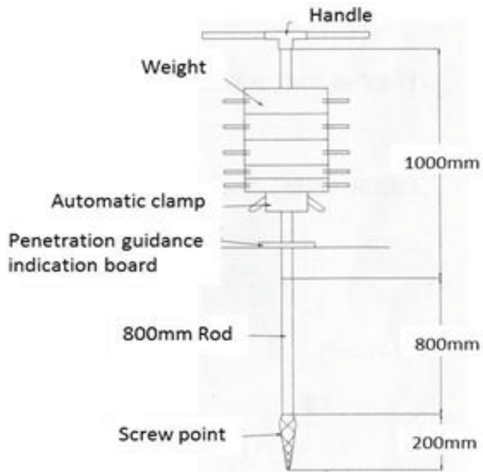

Fig. 2. Swedish Weight Sounding Tool, SWS_Procedure Manual (11 March 2020).

The SWS test kit consists of a screw point (25 mm diameter), a set of loads (1 x 5 kg, 2 x 10 kg, and 3 x 25 kg, total 100 kg), a number of rods with a diameter of 22 mm, and a handle for turning the rods.

Habibi, M. (2006) stated that the drawback of using SWS for liquefaction assessment is the indirect use of test results by converting SWS data into penetration resistance equivalent to N-SPT, this indirect approach reduces the certainty of the test results. To convert the N-SWS value to the N SPT value, the Inada formula is used:

- For Sandy Soil:

$$N = 0.002 \text{ Wsw} + 0.067 \text{ Nsw} \quad (4)$$

- For cohesive soils:

$$N = 0.003 \text{ Wsw} + 0.050 \text{ Nsw} \quad (5)$$

where Wsw is total load (N) and Nsw is number of rounds 180° per 1 meter.

2.7 *Cone Penetration Test*

The Cone Penetration Test (CPT) or more commonly called the sondir test is a static penetrometer that is widely used in Indonesia.

This tool comes from the Netherlands and is known as the Dutch-cone Penetrometer Test. The working principle of this tool is to press the tip of the penetrometer (conus) down with a pressing machine that is anchored to the ground.

This cone is a cone with an angle of 60° with a cross-sectional area of 10 cm^2, which is mounted on a series of inner handlebars and outer casing. In the standard type, the measurement result is only the tip resistance (cone value). This is obtained by pressing only on the inner handlebar. The force required to press the tip of the cone is measured by a pressure gauge mounted on the pressing machine. Measurements are carried out at certain predetermined depths which are usually carried out every 20 cm depth.

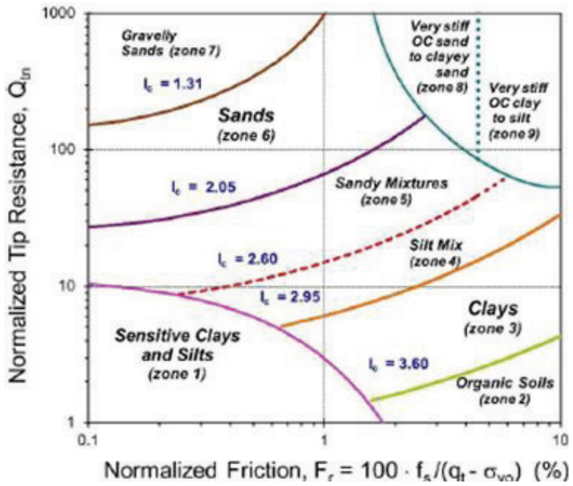

Fig. 3. Graph of Soil Type Based on CPT Test Results, Mayne (2014).

To determine the type of soil, the graph proposed by Robertson is generally used Fig. 3. Where this graph uses the basic value of cone resistance (qt) and friction ratio (Rf). However, because the value of cone resistance and blanket resistance increases with increasing depth, sondir data must be normalized to the value of overburden pressure, Robertson (1990).

$$Qtn = \frac{(qt - \sigma vo)/\sigma atm}{(\sigma vo/\sigma atm)^n} \quad (6)$$

where Qtn is normalized cone resistance, qt is corrected cone resistance (kN/m^2), σvo is overburden pressure (kN/m^2) and σatm is atmospheric pressure = 100 kN/m^2.

Soil classification using Fig. 4 uses the parameter Ic as the zone boundary contained in the table. Ic is the index value of the sondir material obtained by Eqn. (7), Mayne (2014).

$$Ic = \sqrt{(3{,}47 - \log Qtn)2 + (1{,}22 + \log Fr)2} \quad (7)$$

Where Ic is soil characteristic index, Otn is normalized cone resistance (kN/m^2), and Fr is normalized friction ratio (%).

The value of soil density can be obtained using the equation proposed by Mayne (2014) in the following equation. The equation is a correction for friction resistance (fs) and soil density (γ).

$$\gamma_t = 12 + 1.5 \ln(fs + 1) \quad (8)$$

where γ_t is total weight of soil (kN/m^3) and fs is blanket resistance (kN/m^2) (γ).

Table 1. Zone and Soil Type Based on Ic, Mayne (2014).

Zone	Soil Type	Ic
1	Sensitive soil, fine	-
2	Organic soil– peat	>3,6
3	Clay – Silty clay	2,95 – 2,60
4	Silt – Clayey silt	2,60 – 2,95
5	Sand – Sandy silt	2,05 – 2,60
6	Sand – Sand to sandy silt	1,31 – 2,05
7	Coarse sand – dense sand	<1,31
8	Dense sand to clayey sand	-
9	Dense , Fine	-

2.8 *Determine Liquefaction Potential*

The basic principle in the evaluation of soil liquefaction is to calculate the two main variables, namely CRR and CSR, then from these two values the factor of safety will be calculated which will later be used as the basis for determining whether or not liquefaction has the potential to occur. The safety factor against liquefaction hazard can be stated as follows:

$$FS = \frac{CRR}{CSR} \quad (9)$$

If FS is smaller than one (FS <1) then there is the potential for liquefaction to occur, if FS is equal to one (FS = 1) then a critical condition occurs and if FS is greater than one (FS > 1) then there is no potential for liquefaction to occur.

Cyclic Resistance Ratio (CRR) is the ability of the soil to withstand liquefaction. The CRR value is the resistance value of a soil layer to cyclic stress (CSR).

The calculation of the CRR value is based on the data from the SWS test that has been converted into Standard Penetration Test (SPT) data. To calculate the cause of liquefaction in clean sand, this equation applies to, M = 7.5 and the effective overburden stress vc = 1 atm.

$$CRR_{M=7.5} = EXP\left(\left(\frac{(N1)60\ cs}{14.1}\right) + \left(\frac{(N1)60\ cs}{126}\right)^2 - \left(\frac{(N1)60\ cs}{23.6}\right)^3 + \left(\frac{(N1)60\ cs}{25.4}\right)^4 - 2.8 \quad (10)$$

Where (N1)60cs is N-SPT corection for cohesive soils and $CRR_{M=7.5}$ is CRR earthquake of 7.5 Mw.

To calculate the CRR with an earthquake magnitude other than 7.5, a correction factor called the Magnitude Scale Factor (MSF) is needed. In this case the equation can be written as follows:

$$CRR_{MW} = CRR_{M=7.5}.\ MSF\ .\ K\sigma \quad (11)$$

where :

$$MSF = 6.9\ EXP\ \frac{-M}{4}\ 0.058 \quad (12)$$

The $CRR_{7.5}$ CPT value can be calculated using the equation, Idriss and Boulanger (2008) as follows:

$$CRR_{M=7.5} = EXP\left(\left(\frac{(Qc1N)Cs}{113}\right) + \left(\frac{(Qc1N)Cs}{1000}\right)^2 - \left(\frac{(Qc1N)Cs}{140}\right)^3 + \left(\frac{(Qc1N)Cs}{137}\right)^4 - 2{,}8 \quad (13)$$

Cyclic Stress Ratio (CSR) which is the cyclic stress caused by seismic loads (earthquakes) and causes liquefaction. For the evaluation of CSR refers to the equation proposed by Seed and Idris (1971) as written in Robertson (1998).

$$CSR = 0{,}65\ \frac{\alpha\ max}{g}\ \frac{\sigma vc}{\sigma vc'}\ rd \quad (14)$$

where σ_{vc} is total vertical stress (kN/m^2), σ'_{vc} is efectif stress (kN/m^2), a_{maks} is max acceleration (m/s^2), g is gravity acceleration (m/s^2) and rd is stress reduction factor.

2.9 *Shallow Foundation Bearing Capacity*

Shallow foundations are foundations that have a depth of less than 3 meters. The calculation of the depth level is based on one-third of the width of the foundation base. Shallow foundations can only be used on soils that are stable, have high bearing capacity, and are hard. In addition, the specifications of the building should not be too high or too large

$$Qu = \frac{qa\ sws\ .\ (b\ .\ l)}{FS} \quad (15)$$

where Qu is ultimate bearing capacity (kN/m^2), qa SWS is compressive strength SWS (kN/m^2), b is width of foundation (m) and FS is factor of safety.

The allowable bearing capacity depends on how large the FS is selected. In general, the value of FS between 2 to 5. If FS = 3, this means that the designed foundation strength is 3 times the maximum bearing capacity, so the foundation is expected to be safe from collapse.

Types of soil that have both components of soil shear strength c and φ, usually consist of a mixture of several types of soil. The obtained values of c and φ can be used to calculate the ultimate bearing capacity by using the general equations for the ultimate bearing capacity that have been studied, Hardiyatmo (1996).

$$Qu = \frac{(1{,}3\ .c\ .Nc) + (\gamma\ .D\ .Nq) + (0{,}4\ .\gamma\ .B\ .N\ \gamma)}{FS} \quad (16)$$

where Qu is ultimate bearing capacity (kN/m), B is width of foundation (m), D is depth of foundation (m), γ is unit weight, FS is factor of safety and Nc, Nq, Nγ is bearing capacity factor.

With the cone resistance value (Qc) that has been obtained from the data from the CPT test, Mayerhoff provides an equation in calculating the bearing capacity of the soil based on the CPT data. Mayerhoff also suggests using the safety factor FS = 3 to get the value of the allowable carrying capacity

$$Qu = Qc\left(\frac{B}{12{,}2}\right)\left(1 + \frac{D}{B}\right) \quad (17)$$

where Qu is ultimate bearing capacity (kN/m^2), Qc is cone resistance (kN/m^2), B is width of foundation (m) and D is depth of foundation (m).

3 METHODOLOGY

3.1 *Flowchart*

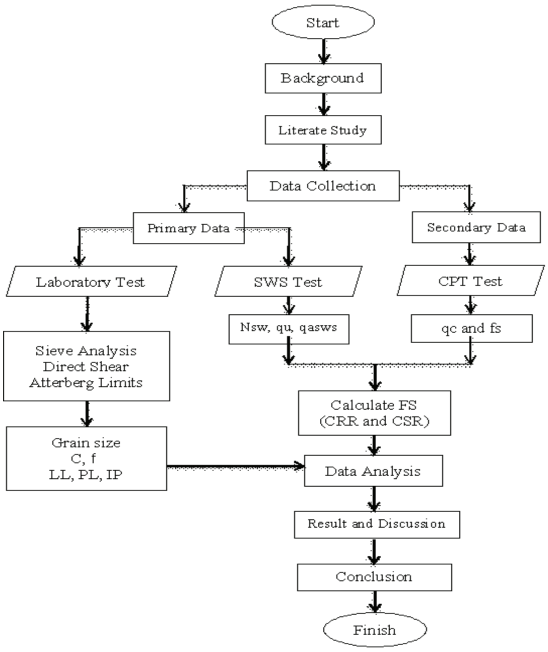

Fig. 4. Research Flowchart.

3.2 *The Location of the Research*

The location of the research and sampling is located at SD Inpres Petobo, Petobo Village, South Palu District, Palu City, Central Sulawesi Province. This school itself is located on the boundary of the liquefaction area, where the distance between this school and the boundary affected by liquefaction is ± 50 m. For more details, the research location can be seen in Fig. 5 and Fig.6.

Fig. 5. Areas Affected by Liquefaction 2018, https://earth.google.com, Feb. 14th 2022.

Fig. 6. Research Locations SDI Petobo, https://earth.google.com, Feb. 14th 2022.

3.3 *Data Collection*

The data collection consists of primary data and secondary data. The primary data obtained is the SWS test where the researcher conducts direct testing in the field as many as 4 test points, then takes soil samples where the sample will go through sieve analysis, Atterberg limits and direct shear in the laboratory. While secondary data consists of CPT obtained from the Faculty of Engineering, Tadulako University

4 RESULTS AND DISCUSSION

4.1 *Sieve Analysis*

The sieve analysis test was carried out first by taking disturbed soil samples in the SD Inpres Petobo area. The samples taken were at a depth of 0.5 m and 0.7 m at each SWS test point.

From the Table 2 of calculation results, it can be seen that the soil samples at points T1 and T2 at each depth are coarse-grained soils of sand type because more than 50% of the fraction passes through sieve no.4. This type of sand has some content of fine grain (SP) with poor gradation.

The sieve analysis test is used to determine the distribution of soil to the grain size distribution curve of the soil that is susceptible to liquefaction by Tsuchida (1970) as shown in Fig. 7.

Table 2. Results of Sieve Analysis.

Point	Depth	% Pass No. 4 Sieve	% Pass No. 200 Sieve	D10	D30	D60	Coefficient of Uniformity Cu	Coefficient of Curvature Cc
	(m)	(%)	(%)	(mm)	(mm)	(mm)		
T1	0.5	94.224	16.645	-	0.120	0.410	-	-
	0.7	94.593	20.489	-	0.110	0.490	-	-
T2	0.5	93.998	8.254	0.080	0.145	0.380	4.750	0.692
	0.7	89.338	11.276	0.075	0.110	0.410	5.467	0.393

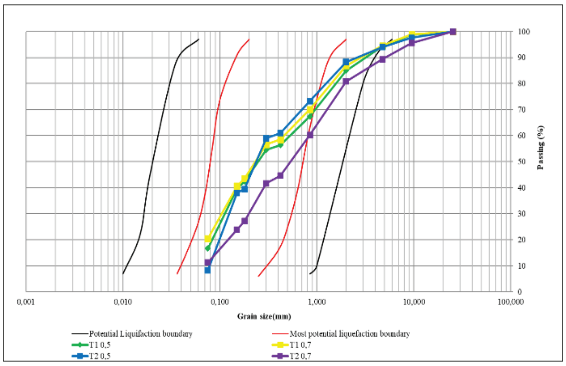

Fig. 7. Graph of the Potential for Liquefaction at Points T1 and T2.

Based on the graph of liquefaction potential according to Tsuchida (1970) it can be concluded that the soil in the research area has the potential for liquefaction to occur, indicated by the results of the test graph that is in the limit of liquefaction potential according to Tsuchida. This could be due to the proximity of the study area to the liquefaction area.

4.2 *Atterberg's Limits*

For the Atterberg Limits test, we use a sieve analysis sample that passes sieve 40. From the conclusion table above, it can be seen that the IP (Plastic Index) value obtained is <7 so that it belongs to the type of clay or silt, low plasticity with partial cohesion. As well as the value of LL (Liquid Limit) which shows a value of <35%, all samples are included in the criteria that have the potential for liquefaction according to Seed et al (1971). And when viewed using the USCS soil classification table on the plasticity diagram, it shows that the soil at both points is included in the ML classification.

Table 3. Result of Atterberg Limits Test.

Point	Depth	Plastic Limit (PL)	Liquid Limit (LL)	Plasticity Index (PI)
	(m)	(%)	(%)	
T1	0.5	21.85	24.12	2.27
	0.7	17.91	22.70	4.79
T2	0.5	21.56	25.93	4.37
	0.7	24.21	27.16	2.94

4.3 *Direct Shear*

Samples for the Direct Shear Test using undisturbed soil were taken at the SWS test point at a depth of 1 m. Sampling for this test uses a pipe with a diameter of 4 inches and a length of 20 cm.

Table 4. Direct Shear Test Results.

Point	Depth	Load	Max. Shear Strength	Cohesion	Shear Angle
	(m)	(kg)	τ Max.	c	Ø
T1	1	3	0.245	0.225	13.534
		6	0.288		
		9	0.296		
T2	1	3	0.266	0.190	35.829
		6	0.344		
		9	0.420		

From the results of the direct shear test presented in the table 4, which was tested at a depth of 1 m, it has a cohesion value of 0.225 kg/cm^2 on the sample at point T1 and a cohesion value of 0.190 kg/cm^2 on the sample at point T2 with a shear angle value for each. each point is 13.534° at sample point T1 and 35.829° at sample point T2.

So, it can be seen that the dominant soil at both test points is dominated by sandy soil types. It can be seen from the high shear angle values found in both samples and the very low cohesion value which indicates that the soil content with a little fine grains.

4.4 *Swedish Weight Sounding*

Field testing of the SWS test was conducted at 2 points which are primary data. The following Fig. 8 and Fig.9 are the parameters to determine the liquefaction potential in the SWS test.

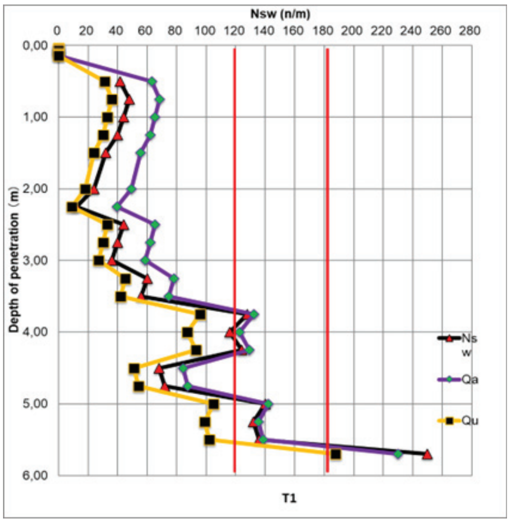

Fig. 8. Graph of the SWS Parameter at Points T1.

From the Fig 8 and Fig.9, can be seen that the highest value for the relationship Nsw, qa, and qu SWS at point T2 is located at a depth of 5.41 m, with the value of Nsw is 304.88 N/m², while the highest value for T1 is located at a depth of 5.70 m. with the value of Nsw is 250N/m².

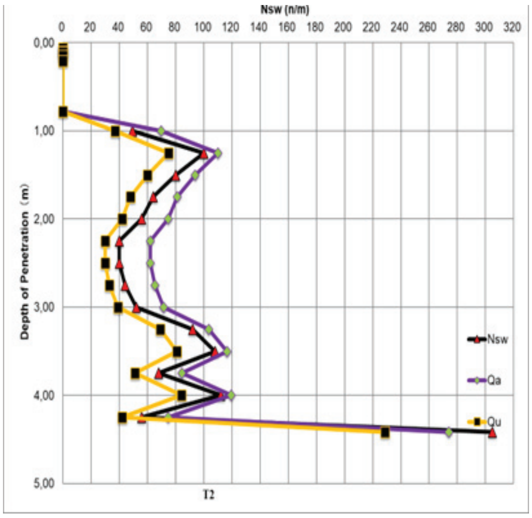

Fig. 9. Graph of the SWS parameter at points T2.

For soils whose Nsw value is more than 180 N/m, the soil is included in the category of soil with low liquefaction potential. For soils that have an Nsw value between 120 N/m - 180 N/m are included in the category of soil with moderate liquefaction potential and if the Nsw value is less than 120 n/m then the soil has a high liquefaction potential.

From the results of the SWS test at both research points, there is a layer of soil that has an Nsw value of more than 180 N/m, T1 = 250 N/m and T2 = 304.88 N/m. The soil that has an Nsw value of more than 180 N/m, are located at the bottom which is a layer that can no longer be penetrated by this SWS drill bit. For soils whose Nsw value is more than 180 N/m, the soil is included in the category of soil with low liquefaction potential. For soils that have an Nsw value between 120 N/m - 180 N/m are included in the category of soil with moderate liquefaction potential and if the Nsw value is less than 120 n/m then the soil has a high liquefaction potential.

The safety factor against liquefaction hazard from SWS test in point T1 discribe ini Table 5.

Table 5. The Potential of Liquefaction at T1.

Depth (m)	N_{SW}	CSR	CRR M=7.5	CRR_{mw}	FS	Desciption
0.04	0.00	0.520	0.083	0.093	0.180	Liquefied
0.05	0.00	0.520	0.085	0.096	0.184	Liquefied
0.07	0.00	0.520	0.087	0.098	0.189	Liquefied
0.11	0.00	0.520	0.093	0.105	0.202	Liquefied
0.13	0.00	0.520	0.099	0.111	0.215	Liquefied
0.14	0.00	0.519	0.105	0/118	0.228	Liquefied
0.50	41.67	0.518	0.123	0.139	0.268	Liquefied
0.75	48.00	0.517	0.126	0.142	0.275	Liquefied
1.00	44.00	0.515	0.124	0.140	0.272	Liquefied
1.25	40.00	0.514	0.122	0.138	0.269	Liquefied
1.50	32.00	0.513	0.199	0.134	0.261	Liquefied
2.00	24.00	0.510	0.115	0.130	0.255	Liquefied
2.25	12.00	0.508	0.110	0.124	0.244	Liquefied
2.50	44.00	0.507	0.124	0.140	0.276	Liquefied
2.75	40.00	0.505	0.122	0.138	0.273	Liquefied
3.00	36.00	0.504	0.120	0.136	0.270	Liquefied
3.25	60.00	0.502	0.132	0.149	0.296	Liquefied
3.50	56.00	0.501	0.130	0.146	0.292	Liquefied
3.75	128.00	0.499	0.181	0.204	0.409	Liquefied
4.00	116.00	0.498	0.171	0.193	0.388	Liquefied
4.25	124.00	0.496	0.177	0.200	0.404	Liquefied
4.50	68.00	0.494	0.137	0.154	0.313	Liquefied
4.75	72.00	0.493	0.139	0.157	0.320	Liquefied
5.00	140.00	0.491	0.191	0.216	0.440	Liquefied
5.25	1312.00	0.489	0.184	0.208	0.425	Liquefied
5.50	136.00	0.487	0.188	0.212	0.435	Liquefied
5.75	250.00	0.486	0.388	0.439	1	No Liquefied

4.5 *Cone Penetration Test*

Field testing of the Cone Penetration Test was carried out at 2 test points which were secondary data. In this journal only the first point T1 is included.

Table 6. Soil Classification Based on CPT.

Depth (m)	Fr (%)	Qtn	Ic	n	Soil Type
0.00	0.00	0.00	0.00	0.00	-
0.20	9.08	87.95	2.66	0.86	Clay
0.40	9.15	54.69	2.79	0.91	Clay
0.60	9.21	40.47	2.87	0.95	Silty clay
0.80	4.57	50.21	2.58	0.84	Clayey sand
1.00	4.59	42.94	2.63	0.86	Silty clay
1.20	3.05	50.05	2.46	0.80	Clayey sand
1.40	3.06	45.06	2.49	0.81	Clayey sand
1.60	2.29	51.01	2.37	0.77	Clayey sand
1.80	2.29	47.15	2.39	0.78	Clayey sand
2.00	2.30	43.86	2.42	0.79	Clayey sand
2.20	2.30	41.01	2.44	0.80	Clayey sand
2.40	2.30	38.51	2.46	0.81	Clayey sand
2.60	1.84	43.92	2.35	0.77	Clayey sand
2.80	1.53	48.81	2.27	0.74	Clayey sand
3.00	1.14	59.95	2.12	0.68	Clayey sand
3.20	1.01	63.90	2.07	0.67	Clayey sand
3.40	1.14	106.58	1.93	0.61	Sand
3.60	0.72	154.39	1.68	0.52	Sand
3.80	0.58	182.98	1.56	0.48	Sand
4.00	0.83	66.51	2.00	0.65	Sand
4.20	0.76	70.08	1.96	0.64	Sand
4.40	0.65	78.95	1.88	0.61	Sand
4.60	0.96	104.49	1.88	0.61	Sand
4.80	0.63	153.52	1.64	0.51	Sand
5.00	0.91	155.90	1.74	0.56	Sand
5.20	0.73	187.47	1.62	0.51	Sand
5.40	0.93	193.58	1.68	0.51	Sand
5.60	1.13	194.18	1.74	0.56	Sand
5.80	1.36	189.60	1.80	0.59	Sand
6.00	1.59	184.91	1.86	0.61	Sand

Table 6 describe the type of soil which is then used as a parameter for calculating the liquefaction potential. From the results of the CPT test research points, it was found that in one soil depth such as at point T1 which has a depth of 6 m, various results were obtained, some were potential and some did not have the potential for liquefaction, in this case what if the soil with no potential for liquefaction was located at above the soil layer with the potential for liquefaction, it can be said that the soil layer has the potential for liquefaction to occur.

Relationship between the cone value (qc) and FS describe in figure 10. If we observe that the cone value (qc) has a straight relationship with the FS value that has been obtained, where the value of FS 1 has an average qc value of > 75 kg/cm^2 at point T1.

4.6 *Soil Bearing Capacity Based on SWS*

The allowable bearing capacity can be calculated with the value of the SWS compressive strength (qu sws). Varying dimensions of shallow foundations, the largest bearing capacity value is obtained based on the SWS parameter with dimensions of 1 m x 1 m at a depth of 1 m with a Qult value of 21.733 kN/m^2 at point T1, Table 7.

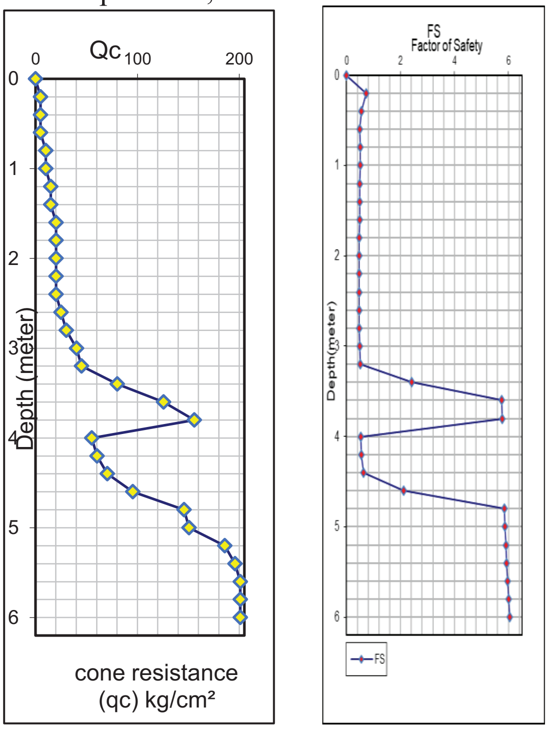

Fig. 10. Graph of the CPT Parameter and FS.

Table 7. Allowable Bearing Capacity by SWS Test.

Depth (m)	Qa SWS (kN/m^2)	Dimensions B (m)	L (m)	Qult (kN/m^2)
		0.6	0.6	43.09
1.0	65.20	0.8	0.8	48.47
		1.0	1.0	53.86
		0.6	0.6	98.96
1.5	55.60	0.8	0.8	108.39
		1.0	1.0	117.81
		0.6	0.6	5.90
2	49.20	0.8	0.8	10.49
		1.0	1.0	16.40

4.7 *Soil Bearing Capacity Based on c and φ*

With the cohesion values and shear angles that have been obtained in the direct shear test, for the calculation of the bearing capacity of the foundation using this method is only limited to a depth of 1 m with several dimensions of the foundation size which can be seen in the Table 8.

Table 8. Foundation Bearing Capacity Based on c and Ø.

Point	Depth (m)	ɣ (kN/m^3)	c (kg/cm^2)	Ø (°)	Nc	Nq	Nɣ	B (m)	Qu (kN/m^3)
T1	1	12.193	0.225	13.534	11.932	3.901	2.119	0.6	29.56
								0.8	30.25
								1	30.94
T2	1	12.021	0.190	35.829	64.086	48.018	52.020	0.6	295.20
								0.8	311.88
								1	328.55

4.8 *Soil Bearing Capacity Based on qc CPT*

With the cone resistance value (qc) that has been obtained from the data from the CPT test results, the bearing capacity of the soil can be calculated with equation 17. The result is in Table 9.

Table 9. Result of Bearing Capacity.

Depth (m)	qc (kN/m^2)	Dimensions B (m)	L (m)	Qult (kN/m^2)
1.0	985.56	0.6	0.6	43.09
		0.8	0.8	48.47
		1.0	1.0	53.86
1.5	1724	0.6	0.6	98.96
		0.8	0.8	108.39
		1.0	1.0	117.81

5 CONCLUSION

Based on the results of field and laboratory tests conducted on several soil samples, it can be concluded as follows:

1. Liquefaction potential

a. The sieve analysis test using a liquefaction chart by Tsuchida shows that the soil samples in the study area have the potential for liquefaction to occur. Based on the Atterberg Limits test, the PI value obtained is < 4 for both types of soil samples, which means that the sample is a sand type with a small amount of silt (SM).

b. From the results of the SWS test where the Nsw value is converted to the N-SPT value and follows the liquefaction requirements if FS < 1, it is found that at a depth of around 5 m, liquefaction has the potential to occur.

c. From the calculation results of the CPT the results obtained at point T1 a depth of 0.2 m - 3.2 m has the potential for liquefaction to occur, a depth of 3.4 m - 3.8 m does not have the potential for liquefaction to occur, at a depth of 4 m – 4.4 m again there is the potential for liquefaction to occur, and at a depth of 4.6 m to 6 m there is no potential for liquefaction to occur. From the results of the CPT , it was found that in one soil depth such as at point T1 which has a depth of 6 m, various results were obtained, some were potential and some did not have the potential for liquefaction, in this case if the soil with no potential for liquefaction was located at above the soil layer with the potential for liquefaction, it can be said that the soil layer has the potential for liquefaction to occur.

d. From the 3 points above, it is concluded that the soil in the research area has the potential for liquefaction to occur with a potential level of medium to high based on the type of soil. However, because the ground water table is very deep from the surface, the potential for liquefaction at SDN Inpres Petobo is on a low potential scale. The location of SD Inpres Petobo is also at the upper side of the Gumbasa irrigation channel.

2. Soil bearing capacity

The calculation of bearing capacity for shallow foundations with three methods, obtain that at a depth of 1 m with dimensions of 1m x 1 m foundation size is sufficient to support simple construction loads. The bearing capacity values obtained were different between the SWS procedure, the cohesion parameters and the shear angle from the direct shear test and from the CPT test. This is due to the different test equipment and parameters. The simple construction of public facilities in this case Elementary school in the area around the Petobo district using shallow foundations is safe to build.

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