International Journal of Innovative Technology and Exploring Engineering (IJITEE)
ISSN: 2278-3075 (Online), Volume-9 Issue-6, April 2020

Liquefaction Hazard Mapping Based on LPI (Liquefaction Potential Index) At Jepara, Indonesia

Rini Kusumawardani, Muhammad Zain Rais

Abstract*: *Liquefaction is a phenomenon of loss of strength of the soil layers caused by earthquake vibration. Liquefaction causes the soil to be in a liquid – like state, especially on sandy soil. Analysis of liquefaction potential was performed by using the semi-empirical method by calculating the Safety Factor (SF) based on Standard penetration Test (SPT) and Cone Penetration test (CPT) data. After the SF value was obtained, then the Liquefaction Potential Index (LPI) was calculated to determine the level of potential liquefaction in the study area to further produce a liquefaction potential map based on the liquefaction potential index. Based on the results of the calculation of the LPI, the level of liquefaction potential in the study area was very low when the earthquake magnitude is 5 Mw because the Liquefaction Potential Index (LPI) = 0. When the earthquake magnitude is 6 Mw, 7 Mw, 8 Mw, and 9 Mw, most of the investigation area has low potential level and there are some points that a high potential level.

Keywords* : *Earthquakes, Liquefaction, Safety Factor, Liquefaction Potential Index

I. INTRODUCTION

Indonesia is one of the countries located in the ring of fire or the Pacific Ring of Fire, which is an area that often experiences earthquakes and volcanic eruptions. Furthermore, its position is located between three active plates of the world. The geology condition of Indonesia and volcanic activity is the most triggering the earthquake that have occurred within the boundaries of Indonesia. The frequency of earthquake with smaller magnitude occurs very regularly caused by the meeting of major tectonic plates in this region. Many researcher was extensively conducted a research point to liquefaction phenomenon. From the the triggering factors until it reach the behavior of soil particle. The liquefaction phenomenon mostly triggering by dynamic loading. In reality, dynamic loading could come from various sources such as earthquake, pile driven, traffic loading, etc [2]-[5]. A number of severe liquefaction was caused by devastating earthquake.

Revised Manuscript Received on April 30, 2020.
***** Correspondence Author
Rini Kusumawardani*, Civil Engineering Department, Universitas Negeri Semarang, Semarang, Indonesia. Email: rini.kusumawardani@mail.unnes.ac.id
Muhammad Zain Rais, department, Civil Engineering Department, Universitas Negeri Semarang, Semarang, Indonesia. Email: mzainrais@gmail.com

Some of Indonesia's worst liquefaction phenomenon trigger by earthquake was occurred in liquefaction Palu, Indonesia 2019. This paper analyzes the potential liquefaction occurrence at the vital location of the largest steam power plant in Indonesia.

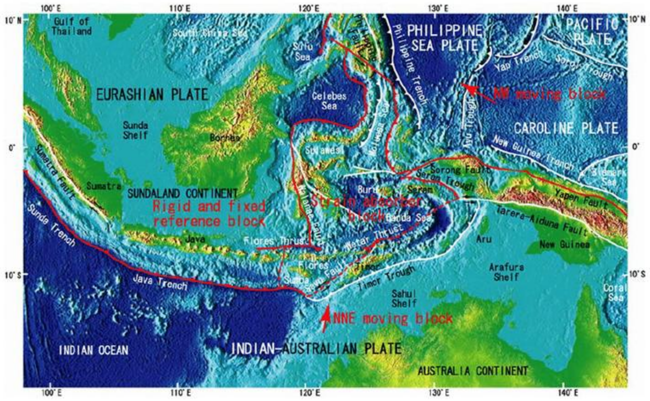

Fig. 1. Plate tectonic surrounding Indonesia [1]

According to Tjasyono [6], earthquakes was defined as a movements or vibration in the earth's crust caused by endogenous energy. One of the problems caused by the earthquake is the severity level of liquefaction disaster. Liquefaction is a phenomenon of loss of strength of the soil layer caused by earthquake loading. Behavior of soil change dramatically from solid to be liquid. It usually occurs in loose sandy soils in saturated condition [7]-[9]. Various factors could be a triggering soil liquefaction phenomenon, i.e earthquake magnitude, ground motion duration and intensity, soil type and soil deposit thickness and physical soil properties. This is condition certainly very dangerous because it could trigger the collapse of a building due to loss of stability of the soil when liquefaction come [9]. Therefore, it is necessary to analyze the liquefaction

II. LITERATURE REVIEW AND THEOROTICAL BASIS

A. Calculation of Safety Factor

The analysis of liquefaction potential requires a determinant whether the liquefaction occurred or not. This determinant is called Safety Factor (FS). Safety Factor (FS) analysis requires the analysis of Cyclic Stress Ratio (CSR) and Cyclic Resistance Ratio (CRR) expressed in the following equation:

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

If $FS = \frac{CRR}{CSR} < 1$ (liquefaction occurs)

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

$FS = \frac{CRR}{CSR} > 1$ (no liquefaction)

B. Calculation of Cyclic Stress Ratio (CSR)

Cyclic Stress Ratio is the cyclic stress caused by an earthquake divided by the effective stress. Seed and Idriss [10] formulated the equation for the cyclic stress ratio (CSR), as follows:

$$CSR = \frac{\tau_{av}}{\sigma'_{vc}} = 0,65 \left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_{vc}}{\sigma'_{vc}}\right) r_d \quad (2)$$

with a_{max} is earthquake acceleration in bedrock (gal) and g is gravity acceleration (gal); σ_{vc} is total vertical stress when consolidating (kN/m^2); σ'_{vc} is effective vertical stress when consolidating (kN/m^2) and r_d is shear stress reduction coefficient.

The factor 0.65 is the assumption that the equivalent uniform shear stress is 65% of the absolute maximum shear stress produced by the earthquake. Earthquake acceleration at the bedrock is determined by using a peak acceleration map in the bedrock for each region of the earthquake in Indonesia.

The ratio of total stress to effective stress is calculated by the equations in the theory of soil mechanics [11].

The formula for finding total stress:

$$\sigma = \gamma . H \quad (3)$$

with,

σ = total soil stress (kN/m^2);

γ = volume weight of soil layer (kN/m^3);

H = thickness of soil layer (m).

The higher effective stress of a soil means denser soil. According to Towhata [12], liquefaction occurs in sandy soil which is not dense and saturated with water. The effective stress of the soil is calculated by:

$$\sigma' = \sigma - \mu \quad (4)$$

where μ is the soil pore water pressure, which is calculated by the equation:

$$\mu = H.\ \gamma_w \quad (5)$$

where H is the point distance observed from the ground water level. The shear stress reduction coefficient is a value that can affect stresses in the soil. The farther depth of the soil leads to the smaller reduction factor. rd proposed by Idriss [13] is as follows:

$$r_d = exp(\alpha(z) + \beta(z)M) \quad (6)$$

$$\alpha(z) = -1{,}012 - 1{,}126 \sin\left(\frac{z}{11{,}73} + 5{,}133\right) \quad (7)$$

$$\beta(z) = 0{,}106 - 0{,}118 \sin\left(\frac{z}{11{,}28} + 5{,}142\right) \quad (8)$$

with M is earthquake magnitude (Mw) and is the depth of soil layer (m).

C. Calculation of Cyclic Resistance Ratio

Cyclic Resistance Ratio (CRR) is the value of the resistance of a soil layer to the cyclic stress.

a. Standar Penetration Test (SPT)

Determine the value of N_{60}

N_{60} is the N-SPT value when the energy ratio is 60% which can be determined using the following formula [14]:

$$N_{60} = N_m\, C_E\, C_B\, C_R\, C_S \quad (9)$$

with,

N_m = N-SPT obtained from the field test;

C_E = correction of hammer energy ratio (ER);

C_B = correction of bore hole diameter;

C_R = correction factor of the rod length;

C_S = correction of samples.

The following are the correction tables used in the SPT test [14]

TABLE 1.
CORRECTION FACTOR VALUES FOR N SPT VALUES

Factor	Description	Parameter	Correction
Energy ratio	ER is a ratio maximum energy in %	C_E	C_E = ER/60
Energy ratio	Donut hammer	C_E	0.5 to 1.0
Energy ratio	Safety hammer	C_E	0.7 to 1.2
Ratio energy	Automatic trip hammer	C_E	0.8 to 1.3
Drill diameter	65 to 115 mm	C_B	1.00
Drill diameter	150 mm	C_B	1.05
Drill diameter	200 mm	C_B	1.15
Rod's length	< 3 m	C_R	0.75
Rod's length	3 to 4 m	C_R	0.80
Rod's length	4 to 6 m	C_R	0.85
Rod's length	6 to 10 m	C_R	0.95
Rod's length	10 to 30 m	C_R	1.00
Sampling	Standard tube sampler	C_S	1.00
Sampling	Tube with coating	C_S	1.1 to 1.3

Determine the value $(N_1)_{60}$

The corrected resistance value of overburden penetration, $(N_1)_{60}$, is calculated using the overburden correction factor, C_N, as follows [14]:

$$(N_1)_{60} = C_N . N_{60} \quad (10)$$

Equation 10 only applies to non-cohesive soils.

This C_N value proposed by Liao and Whitman [15] is expressed in the following equation:

$$CN = \left(\frac{Pa}{\sigma'vc}\right)^{0,5} \leq 1{,}7 \quad (11)$$

With P_a is pressure at 1 atm = 101 kN/m^2 and the value of C_N must not exceed 1,7.

Determine the value $(N_1)_{60CS}$

The value of $(N_1)_{60CS}$ can be determined by using the following equation [14] :

$$(N_1)_{60CS} = (N_1)_{60} + \Delta(N_1)_{60} \quad (12)$$

$$\Delta(N_1)_{60} = \exp\left(1{,}63 + \frac{9{,}7}{FC+0{,}01} - \left(\frac{15{,}7}{FC+0{,}01}\right)^2\right) \quad (13)$$

with FC is Value of *Fines Content* (%). Equations 12 and 13 only apply to non-cohesive soils.

Determine a value of $CRR_{7,5}$

CRR values on earthquake scale (Mw) 7,5 ($CRR_{7,5}$) can be determined using the following equation [14]:

$$CRR_{7,5} = exp\left(\frac{(N_1)_{60CS}}{14,1} + \left(\frac{(N_1)_{60CS}}{126}\right)^2 - \left(\frac{(N_1)_{60CS}}{126}\right)^3 + \left(\frac{(N_1)_{60CS}}{126}\right)^4 - 28\right) \quad (14)$$

If $(N1)_{60CS}$ > 37,5 then the soil does not need to be evaluated because the value is vulnerable to liquefaction, namely when (N1)60CS < 37,5. Jika (N1)60CS > 37,5 the soil is able to withstand seismic loads which can be represented by a CRR value of 7,5 = 2. Equation 14 only applies to non-cohesive soils.

Determine Magnitude Scaling Factors (MSF)

An earthquake with Mw = 7,5 is stated as a reference earthquake by Youd and Idriss [16] so it needs to be corrected for earthquakes with magnitudes smaller or greater than 7,5. According to Idriss [13], the magnitude of the MSF value for earthquakes other than 7,5 Mw can be determined using the following equation:

$$MSF = 6,9\ exp\left(\frac{-M}{4}\right) - 0,058 \quad (15)$$

With M is earthquake magnitude (Mw).

Determine the overburden correction factor (Kσ)

Overburden correction factor values, Kσ can be determined using the following equation [14]:

$$K\sigma = 1 - C\sigma\ ln\left(\frac{\sigma'_{vc}}{Pa}\right) \leq 1,1 \quad (16)$$

with Pais pressure at 1 atm equal to 101 kN/m^2.

The Cσ value can be determined using the following equation (I.M Idriss and RW Boulanger, 2008):

$$C\sigma = \frac{1}{18,9 - 2,55\sqrt{(N1)60}} \leq 0,3 \quad (17)$$

Determine CRR_{Mw}

To calculate the corrected CRR with earthquake magnitude other than 7,5 (CRR_{Mw}) can be calculated by using the following equation [14]

$$CRR_{Mw} = CRR_{7,5}\ .\ MSF.\ K\sigma \quad (18)$$

with MSF is magnitude Scaling Factors and K_σ is overburden correction factor. Equation 18 only applies to non-cohesive soils.

b. Cone Penetration Test (CPT)

For CPT (Cone Penetration Test), the data which become the reference is cone tip resistance (q_c). The steps to obtain a CRR (Cyclic Resistance Ratio) value from the CPT (Cone Penetration Test) data are as follows:

Calculates the value of q_{c1N}

Calculate the value of q_{c1N}, which is the value of the corrected cone tip resistance by using the following equation [16] :

$$q_{c1N} = C_N\ (q_c / Pa) \quad (19)$$

with,

C_N = overburden correction factor;

Pa = pressure at 1 atm = 101 kN/m^2;

q_c = cone tip resistance (kN/m^2).

This C_N value proposed is expressed in the following equation:

$$C_N = \left(\frac{Pa}{\sigma'_{vc}}\right)^{0,5} \leq 1,7 \quad (20)$$

with,

Pa = pressure at 1 atm = 101 kN/m^2.

Calculates the value of Ic

The index value of soil type behavior (Ic) is calculated by using the following equation [17] :

$$Ic = [3,47 - \log Q)^2 + (\log F + 1,22)^2]^{0,5} \quad (21)$$

Calculate the value of Q

Calculate the value of the dimension of the tip resistance (Q) for a suitable clean sand exponent of 0.5 [17]:

$$Q = [(q_c - \sigma_{vc}) / Pa][(Pa / \sigma'_{vc})^n] \quad (22)$$

For clean sand, the exponent value of n = 0.5 and the value of n between 0.5 - 1 for silt and silty sand and for exponent n = 1 is the appropriate value for clay.

Calculate the value of F

The friction resistance dimension value (F) is calculated by using the equation [17] :

$$F = [\ f_s / (q_c - \sigma_{vc})]\ x\ 100\% \quad (23)$$

with f_s = friction resistance (kN/m^2) and q_c = cone tip resistance (kN/m^2).

Calculating the value (qc1N)cs

Calculation of the equivalent value of CPT normalization $(q_{c1N})_{cs}$ can be determined from the following equation:

$$(q_{c1N})_{cs} = q_{c1N} + \Delta\ _{qc1N} \quad (24)$$

$$\Delta(qc1N) = \left(5,4 + \frac{qc1N}{16}\right).\ exp\left(1,63 + \frac{9,7}{FC+0,01} - \left(\frac{15,7}{FC+0,01}\right)^2\right) \quad (25)$$

with FC is Value of *Fines Content* (%). Equations 25 and 26 only apply to non-cohesive soil.

Calculating a value of $CRR_{7,5}$

CRR values on earthquake scale (Mw) 7,5 ($CRR_{7,5}$) can be determined by using the following equation (I.M Idriss and RW Boulanger, 2008):

$$CRR_{7,5} = exp\left(\frac{(qc1N)cs}{540} + \left(\frac{(qc1N)cs}{67}\right)^2 - \left(\frac{(qc1N)cs}{80}\right)^3 + \left(\frac{(qc1N)cs}{114}\right)^4 - 3\right) \quad (26)$$

If $(q_{c1N})_{cs}$ > 211 the land does not need to be evaluated because the value is vulnerable to liquefaction, when $(q_{c1N})_{cs}$ < 211. Jika $(q_{c1N})_{cs}$ > 211 then the soil has sufficient strength to withstand seismic loads which can be represented by a value CRR 7,5 = 2. Equation 27 only applies to non-cohesive soils.

Determine Magnitude Scaling Factors (MSF)

An earthquake with Mw = 7,5 is stated as a reference earthquake by Youd and Idriss, 2001. Therefore. it needs to be corrected for earthquakes with magnitudes smaller or greater than 7,5. According to Idriss [13], the magnitude of the MSF value for earthquakes other than 7,5 Mw can be determined using the following equation:

$$MSF = 6,9\ exp\left(\frac{-M}{4}\right) - 0,058 \quad (27)$$

with M is earthquake magnitude (Mw).

Determine the overburden correction factor (Kσ)

Overburden correction factor values, Kσ, can be determined using the following equation [14]:

$$K\sigma = 1 - C_\sigma \ln\left(\frac{\sigma'_{vc}}{Pa}\right) \le 1,1 \qquad (28)$$

with,Pa is pressure at 1 atm = 101 kN/m^2. The C_σ value can be determined using the following equation [14]:

$$C\sigma = \frac{1}{37,3-8,27\,(q_{c1N})^{0,264}} \le 0,3 \qquad (29)$$

Determine CRR_{Mw}

The corrected CRR with earthquake magnitude other than 7,5 (CRR_{Mw}) can be calculated using the following equation [14] :

$$CRR_{Mw} = CRR_{7,5} \,.\, MSF.\, K\sigma \qquad (30)$$

with MSF= Magnitude Scaling Factors;
Kσ = overburden correction factor.
Equation 31 only applies to non-cohesive soil.

D. Calculation of the Liquefaction Potential Index (LPI)

The Liquefaction Potential Index (LPI) is the most commonly used method developed by Iwasaki (1984). LPI uses the value of the safety factor (FS) and the soil depth function (w (z)).

Interpolation from the analysis results in contours of zone boundaries with low liquefaction potential, high liquefaction potential (high) and very high liquefaction potential (very high).

The Liquefaction Potential Index (LPI) value can be calculated by using the equation proposed by Iwasaki [18] as follows:

$$LPI = \int_{0m}^{20m} F.w(z)\,dz \qquad (31)$$

with,
LPI = Liquefaction Potential Index Value:
F = Potential liquefaction, determined by the equation:
F = (1 – FS) for FS < 1,and F = 0 for FS > 1:
w(z) = Weighting factor of depth,
w(z) = 10 – 0,5z; where z is the observed depth, up to a maximum of 20 meters.

The following are levels of potential liquefaction based on Liquefaction Potential Index:
LPI = 0, with very low liquefaction potential;
0 < LPI < 5, with low liquefaction potential;
5 < LPI < 15, with high liquefaction potential;
LPI > 15, with very high liquefaction potential

III. RESEARCH METHOD

A. Study Area

The location of the study was Jepara regency, Central Java which located along the North Coast of Java. Jepara area was divided into three types of morphology: alluvium plain, coastal plains and volcano area.

The lowland area in Jepara area was covered more than 40%. Majority of soil types in the study area was sand, clay and silt.

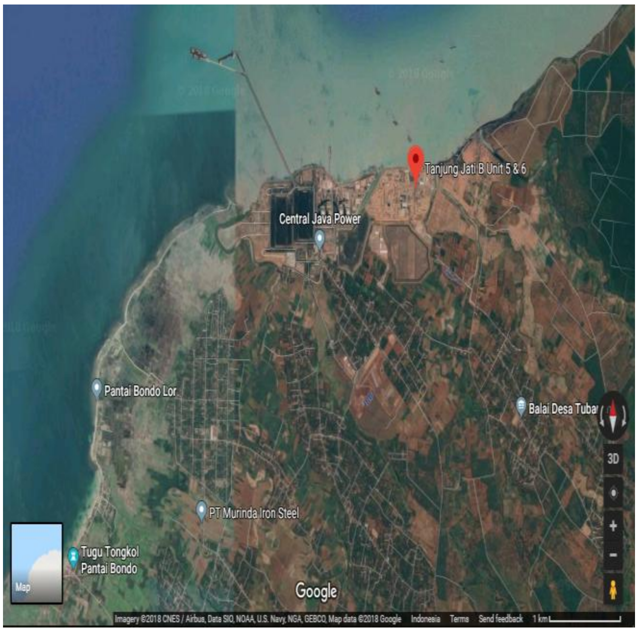

Fig. 2. Location of the Study (Google Maps, 2019)

b. Cone Penetration Test (CPT)

Cone Penetration Test was conducted in the study area for identification for LPI. Some points of CPT were reach more than 15 meters depth.

The result of one CPT points was illustrated in Figure 4. In this area, the soil water level depth was above the soil surface.

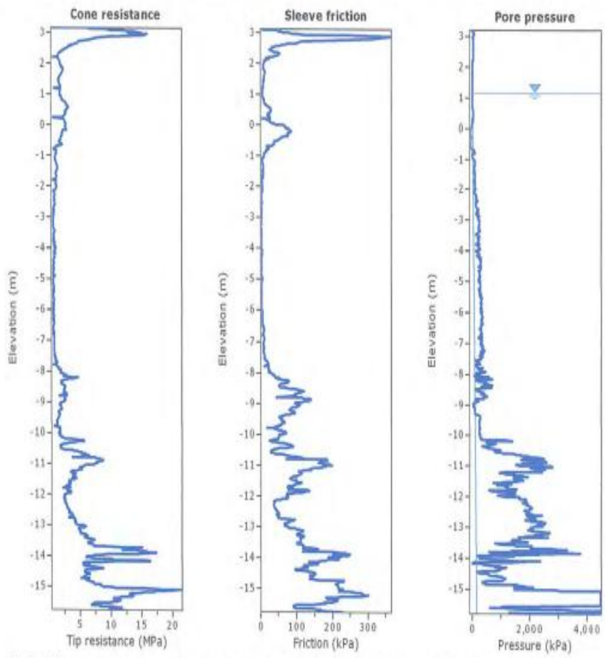

Fig. 3. Field investigation Cone Penetration Test (CPT) test in study area

c. Standard Penetration Test (SPT)

In the field, soil investigation was combined between CPT and SPT. The average depth of borehole was 30 meters depth. From the bore log data, the soft soil was found until 5 meters depth.

After that, the soils was continuously become stiff and very stiff. The illustration of soil type and depth could be seen in Figure 5.

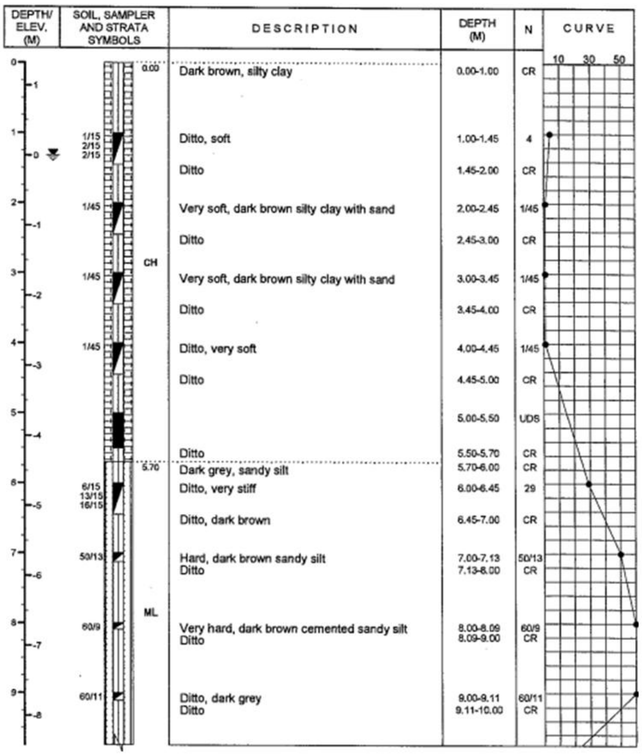

Fig. 4. Field investigation Standard Penetration Test (SPT) test in study area

IV. RESULTS AND DISCUSSION

A. Calculation of Safety Factor

a. Standard Penetration Test (SPT)

From the existing Standard Penetration Test data, it can be seen the potential liquefaction at that point at a certain depth. Values that indicate certain point are safe or not can be determined based on the Safety Factor (FS) value.

TABLE II
RECAPITULATION OF SAFETY FACTOR CALCULATION RESULTS WITH VARIATIONS IN MW VALUES AT ALL SPT POINTS

No	Point	*Safety Factor (FS)*				
		5 Mw	6 Mw	7 Mw	8 Mw	9 Mw
1	PBA-01	1,383	1,095	0,812	0,599	0,443
2	PBA-02	1,111	0,864	0,629	0,456	0,331
3	PBA-03	1,122	0,896	0,670	0,499	0,372
4	PBA-04	1,039	0,823	0,610	0,450	0,333
5	PBA-06	1,155	0,929	0,700	0,525	0,394
6	PBA-07	1,329	1,070	0,806	0,605	0,454
7	PBA-08	2,000	2,000	2,000	2,000	1,929
8	PBA-09	1,090	0,883	0,670	0,506	0,383
9	PBA-10	2,000	2,000	2,000	2,000	2,000
10	PBA-11	2,000	2,000	2,000	1,505	1,022
11	PBA-12	1,053	0,834	0,618	0,457	0,338
12	PBA-14	1,158	0,932	0,702	0,527	0,396
13	PBA-18	2,000	1,563	1,093	0,761	0,529
14	PBA-20	2,000	1,613	1,151	0,819	0,582
15	PBA-21	2,000	2,000	2,000	1,509	1,049
16	PBA-22	1,061	0,833	0,612	0,448	0,328
17	PBA-23	1,122	0,903	0,680	0,511	0,383
18	PBA-24	1,106	0,896	0,680	0,514	0,388
19	PBA-25	1,408	1,149	0,877	0,667	0,507
20	PBA-26	1,936	1,431	0,990	0,683	0,469
21	PBA-27	1,092	0,841	0,606	0,436	0,313
22	PBA-29	1,131	0,903	0,675	0,503	0,375
23	PBA-30	1,224	0,943	0,680	0,488	0,351
24	PBA-33	1,083	0,865	0,646	0,481	0,359
25	PBA-35	1,088	0,862	0,639	0,472	0,349
26	PBA-37	1,225	0,986	0,742	0,557	0,418
27	PBA-38	2,000	1,576	1,079	0,736	0,500
28	PBA-39	1,123	0,897	0,670	0,499	0,372
29	PBA-46	2,000	2,000	2,000	2,000	2,000
30	PBA-48	2,000	2,000	2,000	2,000	1,818

Based on the table above, of the 30 SPT points that have been analyzed, none of the points has the potential for liquefaction during an earthquake with a magnitude of 5 Mw because there are no safety factor less than 1 which is marked in green. On the other hand, when an earthquake with magnitude

b. Cone Penetration Test (CPT)

From the existing CPT test data, it can be seen that the potential liquefaction at that point at a certain depth. Values that indicate this point are safe or not can be known based on the Safety Factor (FS) value.

TABLE III
RECAPITULATION OF SAFETY FACTOR CALCULATION RESULTS WITH VARIATIONS IN MW VALUES AT ALL CPT POINTS

No	Point	*Safety Factor (FS)*				
		5 Mw	6 Mw	7 Mw	8 Mw	9 Mw
1	PBA-04	1,131	0,867	0,622	0,444	0,318
2	PBA-05	1,074	0,835	0,608	0,441	0,320
3	PBA-13	1,029	0,808	0,594	0,435	0,319
4	PBA-15	1,043	0,843	0,637	0,480	0,362
5	PBA-16	1,114	0,889	0,665	0,495	0,369
6	PBA-17	1,027	0,820	0,613	0,456	0,340
7	PBA-19	1,068	0,868	0,661	0,501	0,380
8	PBA-28	1,065	0,820	0,591	0,425	0,305
9	PBA-31	1,031	0,809	0,594	0,435	0,319
10	PBA-32	1,060	0,817	0,589	0,423	0,304
11	PBA-34	1,037	0,821	0,609	0,450	0,332
12	PBA-36	1,014	0,796	0,585	0,428	0,314
13	PBA-38	1,051	0,793	0,560	0,394	0,278
14	PBA-40	1,106	0,890	0,671	0,503	0,378
15	PBA-41	1,073	0,867	0,655	0,493	0,372
16	PBA-42	1,118	0,900	0,677	0,508	0,382
17	PBA-43	1,040	0,848	0,647	0,492	0,374
18	PBA-44	1,609	1,308	0,995	0,755	0,572
19	PBA-45	1,036	0,821	0,608	0,449	0,332
20	PBA-47	1,255	1,023	0,781	0,594	0,452
21	PBA-49	1,652	1,343	1,022	0,775	0,588
22	PBA-50	1,267	1,027	0,779	0,588	0,445

Based on the table above, of the 22 CPT points that have been analyzed, none of the points has the potential for liquefaction during an earthquake with a magnitude of 5 Mw because there are no Safety Factor value less than 1 which is marked in green. On the other hand, when an earthquake with a magnitude of 6 Mw, 7 Mw, 8 Mw, and 9 Mw, most points are indicated as having liquefaction potential because the Safety Factor value is smaller than 1 which is marked in red. From the table above it can also be concluded that the greater the earthquake strength the greater the possibility of soil liquefaction due to the FS getting smaller in value.

B. Calculation of LPI

a. Standard Penetration Test (SPT)

From the existing LPI value data it can be seen the level of potential liquefaction at that point at a certain depth.

TABLE IV
THE RECAPITULATION RESULTS OF THE CALCULATION OF LPI VALUES FROM THE SPT POINT

No	Point	*Liquefaction Potential Index (LPI)*				
		5 Mw	6 Mw	7 Mw	8 Mw	9 Mw
1	PBA-01	0,00	0,00	1,32	2,80	3,90
2	PBA-02	0,00	0,82	2,23	3,26	4,01
3	PBA-03	0,00	0,78	2,48	3,76	4,71
4	PBA-04	0,00	1,24	2,73	3,85	4,67
5	PBA-06	0,00	0,57	2,40	3,80	4,84
6	PBA-07	0,00	0,00	1,55	3,16	4,37
7	PBA-08	0,00	0,00	0,00	0,00	0,00
8	PBA-09	0,00	0,99	2,81	4,20	5,25
9	PBA-10	0,00	0,00	0,00	0,00	0,00
10	PBA-11	0,00	0,00	0,00	0,00	0,00
11	PBA-12	0,00	1,16	2,67	3,80	4,64
12	PBA-14	0,00	0,54	2,38	3,79	4,83
13	PBA-18	0,00	0,00	0,00	0,96	1,88
14	PBA-20	0,00	0,00	0,00	0,91	2,09
15	PBA-21	0,00	0,00	0,00	0,00	0,00
16	PBA-22	0,00	1,09	2,52	3,59	4,37
17	PBA-23	0,00	0,77	2,56	3,92	4,93
18	PBA-24	0,00	0,88	2,72	4,13	5,20
19	PBA-25	0,00	0,00	1,11	3,00	4,43
20	PBA-26	0,00	0,00	0,03	1,11	1,86
21	PBA-27	0,00	0,87	2,17	3,10	3,78
22	PBA-29	0,00	0,72	2,44	3,73	4,69
23	PBA-30	0,00	0,31	1,76	2,81	3,57
24	PBA-33	0,00	1,02	2,65	3,89	4,81
25	PBA-35	0,00	0,97	2,53	3,70	4,56
26	PBA-37	0,00	0,11	2,06	3,54	4,65
27	PBA-38	0,00	0,00	0,00	0,79	1,50
28	PBA-39	0,00	0,77	2,47	3,76	4,71
29	PBA-46	0,00	0,00	0,00	0,00	0,00
30	PBA-48	0,00	0,00	0,00	0,00	0,00

Based on the table above, from the 30 SPT points that have been analyzed, the potential level of liquefaction occurring during an earthquake with a magnitude of 5 Mw is very low because the LPI = 0 values which is marked in green. Meanwhile, when an earthquake with a magnitude of 6 Mw, Mw 7, 8 Mw, and 9 Mw, most points have low level of liquefaction potential because the value of LPI is 0 <LP <5 which are marked in yellow. However, at the time of the earthquake with a magnitude of 9 Mw, there are 2 points with high level of liquefaction potential due to the LPI value is 5 <LPI <15 which are marked in orange. From the table above can be concluded that the greater strength of the earthquake leads to higher level of potential liquefaction because LPI value is increasing.

b. Cone Penetration Test (CPT)

From LPI data could be seen the liquefaction potential from various point in certain depth.

TABLE V
THE RESULT RECAPITULATION OF THE LPI VALUE CALCULATION FROM CPT POINT

No	Point	*Liquefaction Potential Index (LPI)*				
		5 Mw	6 Mw	7 Mw	8 Mw	9 Mw
1	PBA-04	0,00	0,70	1,99	2,92	3,58
2	PBA-05	0,00	0,99	2,35	3,35	4,08
3	PBA-13	0,00	0,88	1,94	2,70	3,23
4	PBA-15	0,00	1,30	3,00	4,29	5,27
5	PBA-16	0,00	0,83	2,52	3,79	4,73
6	PBA-17	0,00	1,35	2,91	4,08	4,95
7	PBA-19	0,00	1,15	2,97	4,37	5,43
8	PBA-28	0,00	0,99	2,25	3,16	3.82
9	PBA-31	0,00	1,24	2,64	3,67	4.4
11	PBA-34	0,00	1,25	2,74	3,85	4,67
12	PBA-36	0,00	1,32	2,70	3,72	4,46
13	PBA-38	0,00	0,93	1,98	2,73	3,25
14	PBA-40	0,00	0,88	2,64	3,97	4,98
15	PBA-41	0,00	1,10	2,84	4,18	5,18
16	PBA-42	0,00	0,80	2,58	3,93	4,95
17	PBA-43	0,00	1,37	3,18	4,57	5,63
18	PBA-44	0,00	0,00	0,04	2,15	3,74
19	PBA-45	0,00	1,26	2,74	3,86	4,68
20	PBA-47	0,00	0,00	1,97	3,65	4,93
21	PBA-49	0,00	0,00	0,00	1,97	3,61
22	PBA-50	0,00	0,00	1,88	3,50	4,72

Based on the table above, from the 22 CPT points that have been analyzed, the level of potential liquefaction at the time of an earthquake with a magnitude of 5 Mw is very low because the LPI = 0 values which are marked in green.

Meanwhile, when an earthquake with a magnitude of 6 Mw, Mw 7, 8 Mw, and 9 Mw, most points have low level of liquefaction potential because the value of LPI is 0 <LPI <5 which are marked in yellow.

However, at the time of the earthquake with a magnitude of 9 Mw, there are 4 points with a high level of potential liquefaction due to their LPI values are 5 <LPI <15 which are marked in orange.

From the table above, it can be concluded that the greater strength of the earthquake leads to the higher level of liquefaction potential because the LPI values are increasing.

V. CONCLUSION

Based on analysis of the level of liquefaction potential using the semi-empirical method that in terms of value Liquefaction Potential Index (LPI), the greater value of earthquake magnitude (Mw) leads to the smaller Safety Factor (FS). As a consequence, the value of Liquefaction Potential Index (LPI) is increasing. This means that the greater LPI value leads to the higher level of liquefaction potential.

REFERENCES

1. Muraoka, Hirofumi, Nasution, Asnawir, Simanjuntak, Janes, Dwipa, Sjafra, Takahashi, Masaaki, Takahashi, Hiroshi, Matsuda, Koji, Sueyoshi, Yoshikazu, Geology and Geothermal Systems in the Bajawa Volcanic Rift Zone, Flores, Eastern Indonesia, Proceedings of the World Geothermal Congress, ISBN: 975-98332-0-4, pp. 1370-1382, Antalya, Turkey, 2005
2. Nugroho, U., Kusumawardani, R., Yuniarti, W., and Hilmi, A. S. Analysis of ESAL factor on flexible pavement at Weleri Ring Road, Indonesia. In AIP Conference Proceedings , Vol. 1818, No. 1, pp. 020037. AIP Publishing, March, 2017. https://doi.org/10.1063/1.4976901
3. Kusumawardani, Rini, Nugroho, Untoro, Fansuri, M. H., Mindiastiwi, T., Yuniarti, W., & Hilmi, A. S. The Impact of Vehicle Load Inducing Vibrations on the Subgrade Soil Particle Acceleration. J. Eng. Sci. Technol, 13(6): 1440-1450, 2018.
4. Kusumawardani, R., Nugroho, U., Handayani, S., and Fananda, M. A. The Analysis Liquefaction Phenomenon of The Flexible Pavement using Seismic Monitoring Equipment. IIUM Engineering Journal, 20(1), 70-78, 2019.
5. Dobry, R., Alvarez, L., 1967. Seismic Failures of Chilean Tailing Dams. Journal of the Soil Mechanics and Foundation Division, ASCE, Volume 102(9), pp. 909–927

6. Tjasyono H.K., Bayong. Meteorologi Indonesia Volume II. (Badan Meteorologi dan Geofisika, 2006).
Tohwata, Ikuo. (2008). Geotechnical Earthquake Engineering. Springer-Verlag Berlin Heidelberg.
7. Chang, M., Kuo, C. P., Shau, S. H., & Hsu, R. E. Comparison of SPT-N-based analysis methods in evaluation of liquefaction potential during the 1999 Chi-chi earthquake in Taiwan. Computers and Geotechnics, 38(3), 393-406, 2011
8. Kusumawardani, R.,Suryolelono, K.B., Suhendro, B., Rifai, AThe Dynamic Response of Unsaturated Clean Sand at A Very Low Frequancy. International Journal of Technology, Vol. 7, No. 1: 123-131, January 2016. doi:10.14716/ijtech.v7i1.1163
9. Seed, H.B. and Idriss, I.M. Simplified Procedure for Evaluation Soil Liquifaction Potential. Journal of Soil Mechanics and Foundation, Division. ASCE, vol.97, No.9:1249 – 1273. 1971
10. Das, Braja M. *Mekanika Tanah (Prinsip-prinsip Rekayasa* Geoteknis) Jilid I. Jakarta: Erlangga. 1993
11. Towhata, I. Geotechnical Earthquake Engineering. Springer Science & Business Media, Berlin.2008
12. Idriss, I.M. An update to the Seed-Idriss simplified procedure for evaluating liquefaction potential, in Proceedings, TRB Workshop on New Approaches to Liquefaction, Publication No. FHWA-RD-99-165, Federal Highway Administration, January. 1999.
13. Idriss, I. M., Boulanger, R. W. Soil Liquefaction During Earthquakes. EERI . 2008.
14. Liao, S.C., and Whitman, R.V. 1986. Overburden correction factors for SPT in sand, J. Geotechnical Eng. ASCE, 112(3): 373-377.
15. Youd, T. L., Idriss, I. M., dkk. (2001). Liquefaction Resistance of Soils: Summary Report from the 1.Geotechnical and Geoenvironmental Eng, ASCE 127(10): 817-33
16. Robertson, P.K. and Wride, C.E. (1998). Evaluating Cyclic Liquefaction Potential Using The Cone Penetration Test. Canadian Geotechnical Journal, 35:442-459
17. Iwasaki, T., Arakawa, T., and Tokida, K. Simplified Procedures for Assessing Soil Liquefaction during Earthquakes. Soil Dynamics and Earthquake Engineering. 3: 49-58. 1984.

AUTHORS PROFILE

Rini Kusumawardani, Lecturer at Civil Engineering Department, Universitas Negeri Semarang. Her research focused on geotechnical field, The big interest research on geohazard risk management.

Muhammad Zain Rais, he is student of Civil Engineering, Universitas Negeri Semarang