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 have 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 triggered by dynamic loading. In reality, dynamic loading could come from various sources such as earthquake, pile driven, traffic loading, etc. A number of severe liquefactions was caused by devastating earthquake. Some of Indonesia’s worst liquefaction phenomenon triggered 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.

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:

\[ FS = \frac{CRR}{CSR} \] (1)

If \( FS < 1 \) (liquefaction occurs)

If \( FS = 1 \) (critical condition)

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
Liquefaction Hazard Mapping based on LPI (Liquefaction Potential Index) at Jepara, Indonesia

\[ FS = \frac{CRR}{CSR} > 1 \text{ (no liquefaction)} \]

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:

\[ CSR = \frac{\tau_{c65}}{\sigma_{ve}' - \mu} = 0.65 \left( \frac{\sigma_{ve}}{\sigma_{ve}' - \mu} \right) r_d \]  

(2)

with \( \tau_{c65} \) is earthquake acceleration in bedrock (gal) and \( g \) is gravity acceleration (gal); \( \sigma_{ve}' \) is total vertical stress when consolidating (kN/m\(^2\)); \( \sigma_{ve} \) 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:

\[ \sigma = \gamma \cdot H \]

(3)

with,
\( \sigma \) = total soil stress (kN/m\(^2\));
\( \gamma \) = 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:

\[ \sigma' = \sigma - \mu \]

(4)

where \( \mu \) is the soil pore water pressure, which is calculated by the equation:

\[ \mu = H \cdot \gamma_s \]

(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:

\[ r_d = \exp(\alpha(z) + \beta(z)M) \]

(6)

\[ \alpha(z) = -1.012 - 1.126 \sin \left( \frac{z}{11.72} \right) + 5.133 \]  

(7)

\[ \beta(z) = 0.106 - 0.118 \sin \left( \frac{z}{11.28} \right) + 5.142 \]

(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]:

\[ N_{60} = N_m \cdot C_E \cdot C_B \cdot C_R \cdot C_5 \]

(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;
\( C_5 \) = correction of samples.

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

<table>
<thead>
<tr>
<th>Factor</th>
<th>Description</th>
<th>Parameter</th>
<th>Correction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Energy ratio</td>
<td>ER is a ratio maximum energy in %</td>
<td>C_E</td>
<td>C_E = ER/60</td>
</tr>
<tr>
<td>Energy ratio</td>
<td>Donut hammer</td>
<td>C_E</td>
<td>0.5 to 1.0</td>
</tr>
<tr>
<td>Energy ratio</td>
<td>Safety hammer</td>
<td>C_E</td>
<td>0.7 to 1.2</td>
</tr>
<tr>
<td>Energy ratio</td>
<td>Automatic trip hammer</td>
<td>C_E</td>
<td>0.8 to 1.3</td>
</tr>
<tr>
<td>Drill diameter</td>
<td>65 to 115 mm</td>
<td>C_B</td>
<td>1.00</td>
</tr>
<tr>
<td>Drill diameter</td>
<td>150 mm</td>
<td>C_B</td>
<td>1.05</td>
</tr>
<tr>
<td>Drill diameter</td>
<td>200 mm</td>
<td>C_B</td>
<td>1.15</td>
</tr>
<tr>
<td>Rod’s length</td>
<td>&lt; 3 m</td>
<td>C_R</td>
<td>0.75</td>
</tr>
<tr>
<td>Rod’s length</td>
<td>3 to 4 m</td>
<td>C_R</td>
<td>0.80</td>
</tr>
<tr>
<td>Rod’s length</td>
<td>4 to 6 m</td>
<td>C_R</td>
<td>0.85</td>
</tr>
<tr>
<td>Rod’s length</td>
<td>6 to 10 m</td>
<td>C_R</td>
<td>0.95</td>
</tr>
<tr>
<td>Rod’s length</td>
<td>10 to 30 m</td>
<td>C_R</td>
<td>1.00</td>
</tr>
<tr>
<td>Sampling</td>
<td>Standard tube sampler</td>
<td>C_5</td>
<td>1.00</td>
</tr>
<tr>
<td>Sampling</td>
<td>Tube with coating</td>
<td>C_5</td>
<td>1.1 to 1.3</td>
</tr>
</tbody>
</table>

Determine the value of \( (N_{60})_{l60} \)

The corrected resistance value of overburden penetration, \( (N_{60})_{l60} \), is calculated using the overburden correction factor, \( C_N, \) as follows [14]:

\[ (N_{60})_{l60} = C_N \cdot N_{60} \]

(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:

\[ C_N = \left( \frac{P_a}{P_s} \right)^{0.5} \leq 1.7 \]

(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 of \( (N_{60})_{l60CS} \)

The value of \( (N_{60})_{l60CS} \) can be determined by using the following equation [14]:

\[ (N_{60})_{l60CS} = (N_{60})_{l60} + \Delta(N_{60})_{l60} \]

(12)

\[ \Delta(N_{60})_{l60} = \exp \left( 1.63 + \frac{9.7}{F_C + 0.01} - \left( \frac{15.7}{F_C + 0.01} \right)^2 \right) \]

(13)

with \( F_C \) 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]:

\[ CRR_{7,5} = \exp \left( \frac{(N_{60})_{l60CS}}{14.1} + \left( \frac{(N_{60})_{l60CS}}{126} \right)^2 - \left( \frac{(N_{60})_{l60CS}}{126} \right)^3 + \left( \frac{(N_{60})_{l60CS}}{126} \right)^4 - 2B \right) \]

(14)

If \( (N_{60})_{l60CS} > 37.5 \) then the soil does not need to be evaluated because the value is vulnerable to liquefaction.
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:

\[
MSF = 6.9 \exp \left( -\frac{M}{4} \right) - 0.058
\]

(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]:

\[
Kσ = 1 - Cσ \ln \left( \frac{\sigma_{cs}}{\sigma_c} \right) \leq 1.1
\]

(16)

with, \( \sigma_c \) is pressure at 1 atm equal to 101 kN/m².

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

\[
Cσ = \frac{1}{19.9 - 1.55 \sqrt{(N1)60}} \leq 0.3
\]

(17)

**Determine CRR\(_{\text{MSF}}\)**

To calculate the corrected CRR with earthquake magnitude other than 7.5 (CRR\(_{\text{MSF}}\)) can be calculated by using the following equation [14]:

\[
\text{CRR}_{\text{MSF}} = \text{CRR}_{7.5} \times \text{MSF} \times Kσ
\]

(18)

with MSF is magnitude Scaling Factors and Kσ 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 are cone tip resistance (qc). The steps to obtain a CRR (Cyclic Resistance Ratio) value from the CPT (Cone Penetration Test) data are as follows:

**Calculates the value of \( q_{\text{c1N}} \)**

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

\[
q_{\text{c1N}} = C_N (q_c / Pa)
\]

(19)

with, \( C_N \) is overburden correction factor; \( Pa \) = pressure at 1 atm = 101 kN/m²; \( q_c \) = cone tip resistance (kN/m²).

This \( C_N \) value proposed is expressed in the following equation:

\[
C_N = \left( \frac{Pa}{\sigma_{cs}} \right)^{0.5} \leq 1.7
\]

(20)

with, \( Pa \) = pressure at 1 atm = 101 kN/m².

**Calculates the value of \( Ic \)**

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

\[
Ic = [3.47 - \log Q] + (\log F + 1.22)^2]^{0.5}
\]

(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]:

\[
Q = \left( \frac{q_c - \sigma_{oc}}{Pa} \right) \left( \frac{\sigma_c}{\sigma'_{oc}} \right)^n
\]

(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]:

\[
F = \frac{f_c}{(q_c - \sigma_{oc})} \times 100\%
\]

(23)

with \( f_c \) = friction resistance (kN/m²) and \( q_c \) = cone tip resistance (kN/m²).

**Calculating the value (qc1N)cs**

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

\[
(q_{\text{c1N}})_{\text{cs}} = q_{\text{c1N}} + \Delta q_{\text{c1N}}
\]

(24)

\[
\Delta(q_{\text{c1N}})_{\text{cs}} = \left[ \frac{5.4 + 2c_{\text{fc}}}{40} \right] \exp(1.63 - \frac{0.7}{F+0.01}) + \left[ \frac{18.7}{3} \right] \exp \left( \frac{2.46}{F+0.01} \right)
\]

(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):

\[
\text{CRR}_{7.5} = \exp \left( \frac{\sigma_{oc}}{\sigma_{oc}} \right) + \left( \frac{\sigma_{oc}}{\sigma_{oc}} \right)^{2} - \left( \frac{\sigma_{oc}}{\sigma_{oc}} \right)^{3} + \left( \frac{\sigma_{oc}}{\sigma_{oc}} \right)^{4} - 3
\]

(26)

If \( (q_{\text{c1N}})_{\text{cs}} > 211 \) the land does not need to be evaluated because the value is vulnerable to liquefaction, when \( (q_{\text{c1N}})_{\text{cs}} < 211 \) Jika \( (q_{\text{c1N}})_{\text{cs}} > 211 \) 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:

\[
MSF = 6.9 \exp \left( -\frac{M}{4} \right) - 0.058
\]

(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]:

\[
Kσ = 1 - Cσ \ln \left( \frac{\sigma_{cs}}{\sigma_c} \right) \leq 1.1
\]

(28)
\[
C \sigma = \frac{1}{27.3 - 8.27 (\text{CRI} / 10^{0.264})} \leq 0.3 
\]  

(29)

**Determine CRR\(_{Mw}\)**

The corrected CRR with earthquake magnitude other than 7.5 (CRR\(_{Mw}\)) can be calculated using the following equation [14]:

\[
CRR_{Mw} = CRR_{7.5} \cdot MSF \cdot K_{\sigma} 
\]

(30)

with MSF = Magnitude Scaling Factors;

\(K_{\sigma}\) = 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:

\[
LPI = \int_{0}^{20m} F \cdot w(z) \, dz 
\]

(31)

with,

LPI = Liquefaction Potential Index Value:

\(F\) = Potential liquefaction, determined by the equation:

\[F = (1 - FS)\text{ for } FS < 1, \text{ and } F = 0 \text{ for } FS > 1:\]

\(w(z)\) = 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 were sand, clay and silt.

**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.

**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.
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

<table>
<thead>
<tr>
<th>No</th>
<th>Point</th>
<th>Safety Factor (FS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>PBA-04</td>
<td>1.131</td>
</tr>
<tr>
<td>2</td>
<td>PBA-05</td>
<td>1.074</td>
</tr>
<tr>
<td>3</td>
<td>PBA-13</td>
<td>1.029</td>
</tr>
<tr>
<td>4</td>
<td>PBA-15</td>
<td>1.043</td>
</tr>
<tr>
<td>5</td>
<td>PBA-16</td>
<td>1.114</td>
</tr>
<tr>
<td>6</td>
<td>PBA-17</td>
<td>1.027</td>
</tr>
<tr>
<td>7</td>
<td>PBA-19</td>
<td>1.068</td>
</tr>
<tr>
<td>8</td>
<td>PBA-28</td>
<td>1.065</td>
</tr>
<tr>
<td>9</td>
<td>PBA-31</td>
<td>1.031</td>
</tr>
<tr>
<td>10</td>
<td>PBA-32</td>
<td>1.060</td>
</tr>
<tr>
<td>11</td>
<td>PBA-33</td>
<td>1.037</td>
</tr>
<tr>
<td>12</td>
<td>PBA-36</td>
<td>1.014</td>
</tr>
<tr>
<td>13</td>
<td>PBA-38</td>
<td>1.051</td>
</tr>
<tr>
<td>14</td>
<td>PBA-40</td>
<td>1.106</td>
</tr>
<tr>
<td>15</td>
<td>PBA-41</td>
<td>1.073</td>
</tr>
<tr>
<td>16</td>
<td>PBA-42</td>
<td>1.118</td>
</tr>
<tr>
<td>17</td>
<td>PBA-43</td>
<td>1.040</td>
</tr>
<tr>
<td>18</td>
<td>PBA-44</td>
<td>1.169</td>
</tr>
<tr>
<td>19</td>
<td>PBA-45</td>
<td>1.036</td>
</tr>
<tr>
<td>20</td>
<td>PBA-47</td>
<td>1.255</td>
</tr>
<tr>
<td>21</td>
<td>PBA-49</td>
<td>1.652</td>
</tr>
<tr>
<td>22</td>
<td>PBA-50</td>
<td>1.267</td>
</tr>
</tbody>
</table>

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 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

### 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

<table>
<thead>
<tr>
<th>No</th>
<th>Point</th>
<th>Safety Factor (FS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>PBA-01</td>
<td>1.383</td>
</tr>
<tr>
<td>2</td>
<td>PBA-02</td>
<td>1.111</td>
</tr>
<tr>
<td>3</td>
<td>PBA-03</td>
<td>1.122</td>
</tr>
<tr>
<td>4</td>
<td>PBA-04</td>
<td>1.039</td>
</tr>
<tr>
<td>5</td>
<td>PBA-05</td>
<td>1.155</td>
</tr>
<tr>
<td>6</td>
<td>PBA-07</td>
<td>1.329</td>
</tr>
<tr>
<td>7</td>
<td>PBA-08</td>
<td>2.000</td>
</tr>
<tr>
<td>8</td>
<td>PBA-09</td>
<td>1.909</td>
</tr>
<tr>
<td>9</td>
<td>PBA-10</td>
<td>2.000</td>
</tr>
<tr>
<td>10</td>
<td>PBA-11</td>
<td>2.000</td>
</tr>
<tr>
<td>11</td>
<td>PBA-12</td>
<td>1.053</td>
</tr>
<tr>
<td>12</td>
<td>PBA-14</td>
<td>1.158</td>
</tr>
<tr>
<td>13</td>
<td>PBA-18</td>
<td>2.000</td>
</tr>
<tr>
<td>14</td>
<td>PBA-20</td>
<td>2.000</td>
</tr>
<tr>
<td>15</td>
<td>PBA-21</td>
<td>2.000</td>
</tr>
<tr>
<td>16</td>
<td>PBA-22</td>
<td>1.061</td>
</tr>
<tr>
<td>17</td>
<td>PBA-23</td>
<td>1.122</td>
</tr>
<tr>
<td>18</td>
<td>PBA-24</td>
<td>1.106</td>
</tr>
<tr>
<td>19</td>
<td>PBA-25</td>
<td>1.408</td>
</tr>
<tr>
<td>20</td>
<td>PBA-26</td>
<td>1.936</td>
</tr>
<tr>
<td>21</td>
<td>PBA-27</td>
<td>1.092</td>
</tr>
</tbody>
</table>

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 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 to a certain depth.

**TABLE IV**

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

<table>
<thead>
<tr>
<th>No</th>
<th>Point</th>
<th>Liquefaction Potential Index (LPI)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>5 Mw</td>
</tr>
<tr>
<td>1</td>
<td>PBA-01</td>
<td>0.00</td>
</tr>
<tr>
<td>2</td>
<td>PBA-02</td>
<td>0.00</td>
</tr>
<tr>
<td>3</td>
<td>PBA-03</td>
<td>0.00</td>
</tr>
<tr>
<td>4</td>
<td>PBA-04</td>
<td>0.00</td>
</tr>
<tr>
<td>5</td>
<td>PBA-06</td>
<td>0.00</td>
</tr>
<tr>
<td>6</td>
<td>PBA-07</td>
<td>0.00</td>
</tr>
<tr>
<td>7</td>
<td>PBA-08</td>
<td>0.00</td>
</tr>
<tr>
<td>8</td>
<td>PBA-09</td>
<td>0.00</td>
</tr>
<tr>
<td>9</td>
<td>PBA-10</td>
<td>0.00</td>
</tr>
<tr>
<td>10</td>
<td>PBA-11</td>
<td>0.00</td>
</tr>
<tr>
<td>11</td>
<td>PBA-12</td>
<td>0.00</td>
</tr>
<tr>
<td>12</td>
<td>PBA-14</td>
<td>0.00</td>
</tr>
<tr>
<td>13</td>
<td>PBA-18</td>
<td>0.00</td>
</tr>
<tr>
<td>14</td>
<td>PBA-20</td>
<td>0.00</td>
</tr>
<tr>
<td>15</td>
<td>PBA-21</td>
<td>0.00</td>
</tr>
<tr>
<td>16</td>
<td>PBA-22</td>
<td>0.00</td>
</tr>
<tr>
<td>17</td>
<td>PBA-23</td>
<td>0.00</td>
</tr>
<tr>
<td>18</td>
<td>PBA-24</td>
<td>0.00</td>
</tr>
<tr>
<td>19</td>
<td>PBA-25</td>
<td>0.00</td>
</tr>
<tr>
<td>20</td>
<td>PBA-26</td>
<td>0.00</td>
</tr>
<tr>
<td>21</td>
<td>PBA-27</td>
<td>0.00</td>
</tr>
<tr>
<td>22</td>
<td>PBA-29</td>
<td>0.00</td>
</tr>
<tr>
<td>23</td>
<td>PBA-30</td>
<td>0.00</td>
</tr>
<tr>
<td>24</td>
<td>PBA-33</td>
<td>0.00</td>
</tr>
<tr>
<td>25</td>
<td>PBA-35</td>
<td>0.00</td>
</tr>
<tr>
<td>26</td>
<td>PBA-37</td>
<td>0.00</td>
</tr>
<tr>
<td>27</td>
<td>PBA-38</td>
<td>0.00</td>
</tr>
<tr>
<td>28</td>
<td>PBA-39</td>
<td>0.00</td>
</tr>
<tr>
<td>29</td>
<td>PBA-46</td>
<td>0.00</td>
</tr>
<tr>
<td>30</td>
<td>PBA-48</td>
<td>0.00</td>
</tr>
</tbody>
</table>

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, 7, 8 Mw, and 9 Mw, most points have low level of liquefaction potential because the value of LPI = 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**


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