Evaluation of liquefaction potential using cone penetration test (CPT) and standard penetration test (SPT)

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Abstract. To evaluate soil resistance against liquefaction, a simplified procedure has been developed based on directly field soil testing. There are four recommended field tests, including CPT and SPT. Soil resistance to liquefaction is measured by the safety factor SF, which is the ratio between the capacity of the soil to resist liquefaction cyclic resistance ratio (CRR) and the soil stress occurs due to an earthquake cyclic stress ratio (CSR). If SF <1, liquefaction occurs. This research was carried out at Sanur area, Southeast Denpasar City, Bali, by conducting 6 pairs of CPT and SPT tests, each of 6-meter depth. The Ground Water Level (GWL) at this area is 1.5 meter below the soil surface. The soil type is silty sand to sandy silt, with the unit weight between 1.617 to 1.837 g/cm^3. The calculation results, both with CPT and SPT, show that the soil layer did not experience liquefaction with earthquake magnitude Mw = 4.0. At Mw = 5.0, liquefaction occurs in most soil layers, except the 1.5-meter upper soil layer. On Mw = 6.0, almost all soil layers experience liquefaction. Evaluation of soil resistance to liquefaction using CPT and SPT gives results that are not much different.

1. Introduction

Liquefaction as a result of earthquakes has occurred throughout the world. Some of them are the US Alaska earthquake (1964), Niigata Japan (1964), US Loma-Prieta (1989), Kobe Japan (1995), Chi-Chi Taiwan (1999), Bhuj India (2001), Sulawesi Donggala (2018) and many more [2, 3], [9]. Bali is an area prone to earthquakes. In Indonesian zoning seismic areas, Bali is included in zone 5. As part of the island of Bali, Denpasar City is also an area prone to earthquakes so that the effects of earthquakes, including liquefaction, need to be anticipated. Attention should be given to the dangerous nature of this liquefaction, so it needs to identify areas that have the potential to experience liquefaction. The aim is to provide information to the public and interested parties, to avoid areas that have the potential to experience liquefaction.

2. Literature review

Evaluation of liquefaction potential can be done either through laboratory tests or field tests [9, 10]. To avoid difficulties when taking samples and conducting testing in the laboratory, to evaluate the liquefaction potential, a simplified procedure has been developed based on the results of field tests [8], [10]. Four field tests are recommended, i.e. CPT, SPT, Shear Wave Velocity Measurement Vs dan Becker Penetration Test (BPT). In this research, the CPT and SPT methods will be used. The liquefaction potential is determined by calculating the safety factor SF, which is a ratio between the

ability of the soil to withstand liquefaction CRR (Cyclic Resistance Ratio) with stresses that occur in the soil layer due to the earthquake CSR (Cyclic Stress Ratio), which is expressed by the equation:

$$SF = (CRR/CSR) \tag{1}$$

The soil layer is said to be in a critical condition if SF is equal to one[6]. If SF is less than one, the soil experiences liquefaction.

2.1. Determine CSR
Cyclic Stress Ratio is calculated using the equation :

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

where a_{max} is peak ground acceleration, g is acceleration of gravity, σ_{ov} is overburden stress, σ'_{ov} effective vertical stress and r_d stress reduction factor, calculated using:

$$r_d = \frac{(1.000 - 0.4113z^{0.5} + 0.04052z + 0.001753z^{1.5})}{(1.000 - 0.4177z^{0.5} + 0.05729z - 0.006205z^{1.5} + 0.001210z^2)} \tag{3}$$

where z is the depth below soil surface.

2.2. Evaluation CSR using SPT
Based on SPT test result, CRR is calculated using [4]:

$$CRR_{7.5} = \frac{1}{34 - (N_1)_{60cs}} + \frac{(N_1)_{60cs}}{135} + \frac{50}{[10(N_1)_{60cs} + 45]^2} - \frac{1}{200} \tag{4}$$

where $(N_1)_{60cs}$ is $(N_1)_{60}$ corrected to the influence of finest content to CRR. $(N_1)_{60cs}$ is determined using formula:

$$(N_1)_{60cs} = \alpha + \beta(N_1)_{60} \tag{5}$$

where α and β is determined using the formula:

$$\alpha = 0 \text{ for } FC \leq 5\% \tag{6}$$

$$\alpha = \exp[1.76 - (190/FC^2] \text{ for } 5\% < FC < 35\% \tag{7}$$

$$\alpha = 5.0 \text{ for } FC \geq 35\% \tag{8}$$

$$\beta = 1.0 \text{ for } FC \leq 5\% \tag{9}$$

$$\beta = [0.99 + (FC^{1.5}/1,000] \text{ for } 5\% < FC < 35\% \tag{10}$$

$$\beta = 1.2 \text{ for } FC \geq 35\% \tag{11}$$

$(N_1)_{60}$ is determined by:

$$(N_1)_{60} = N_m C_N C_E C_B C_R C_S \tag{12}$$

where N_m is N-SPT value, C_N correction factor to overburden effective P_a = 100 kPa (1 atm), C_E correction to energy ratio (ER), C_B correction to bore hole diameter, C_R correction to rod length and C_S correction for sampling with or without liner. C_N is determined by the formula given by Liao dan Whitman (1986):

$$C_N = 2.2/(1.2 + \sigma'_{vo}/Pa) \tag{13}$$

2.3. Evaluation CRR using CPT

Based on CPT test, CRR is determined using the formula by Robertson and Wride (1998) [5]:

$$\text{If } (q_{c1N})_{cs} < 50 \text{ then } CRR_{7.5} = 0.833[(q_{c1N})_{cs}/1{,}000] + 0.05 \tag{14}$$

$$\text{If } 50 \leq (q_{c1N})_{cs} < 160 \text{ then } CRR_{7.5} = 0.93[(q_{c1N})_{cs}/1{,}000]^3 + 0.08 \tag{15}$$

where $(q_{c1N})_{cs}$ is normalized tip cone resistance q_c to Pa and corrected to finest content <5% (clean sand). The value of q_c normalized to 1 ATM pressure (q_{c1N}) is determined using formula:

$$q_{c1N} = C_Q(q_c/P_a) \tag{16}$$

where

$$C_Q = (Pa/\sigma'_{vo})^n \tag{17}$$

where C_Q is a factor to normalized tip cone resistance, Pa = 1 atm, n is a factor that depends on soil type (0.5 to 1.0) and q_c is tip cone resistance. At shallow depth, the value of C_Q become large because of low overburden pressure. However, value > 1.7 should not be applied. The influence of soil characteristic to the value of (q_{c1N}) and CRR could be determined using soil behavior type index I_c proposed by Robertson and Wride [10]. I_c is calculated using:

$$Ic = [(3.47 - \log Q)^2 + (1.22 + \log F)^2]^{0.5} \tag{18}$$

where

$$Q = [(q_c - \sigma_{vo})/P_a][(P_a/\sigma'_{vo})^n] \tag{19}$$

and

$$F = [f_s/ (q_c - \sigma_{vo})] \text{ x } 100\% \tag{20}$$

Robertson and Wride provide recommendations for procedures to calculate Ic, firstly consider the type of soil is clay, by entering the value of n = 1 to calculate the amount of the dimensionless cone tip resistance Q, using formula:

$$Q = [(q_c - \sigma_{vo})/P_a][(P_a\sigma'_{vo})^1] = [(q_c - \sigma_{vo})/ \sigma'_{vo}] \tag{21}$$

If I_c >2.6 then the soil is classified as clayey and considered too difficult to liquefy and the analysis is completed. To ensure that the soil layer will not experience liquefaction, it is necessary to take a sample of the soil for further testing in the laboratory or be tested using other criteria. Bray and Sancio (2006) [2] say that soil could experience liquefaction if the ratio of $\omega_n/LL > 0.85$, Plasticity Index (PI) <12, ω_n/LL ratio > 0.8 and PI <18. If from (22) I_c<2.6 soil is likely granular in nature then C_Q and Q should be calculated using n=0.5. Next, Ic is recalculated using new Q value. If from the recalculation the value of I_c<2.6 then the soil is classified as granular and nonplastic, and the value of I_c is then used to determine CRR. But, if recalculated I_c is >2.6 then the soil layer is likely to be very silty and possibly plastic. In such case, (q_{c1N}) should be recalculated using n = 0,7. I_c should also recalculated using the new (q_{c1N}) to determine CRR. The value of $(q_{c1N})_{cs}$ is determined using:

$$(q_{c1N})_{cs} = K_c\, q_{c1N} \tag{22}$$

where K_c is grain characteristic correction factor and is defined by the following equation proposed by Robertson and Wride (1998):

$$\text{for } I_c \leq 1.64 \text{ then } K_c = 1.0 \tag{23}$$

$$\text{for } I_c < 1.64 \ K_c = -0.403 I_c^4 + 5.581\, I_c^3 - 21.63\, I_c^2 + 33.75 I_c - 17.88 \tag{24}$$

2.4. Magnitude Scaling Factor (MSF)

The process to determine CRR describe above is valid only for earthquake magnitude $M_w = 7.5$ $(CRR_{7.5})$ [7], [9]. For M_w other than 7.5, $CRR_{7.5}$ should be corrected by Magnitude Scaling Factor

MSF. This applies, both for CPT and SPT. For M_w <7.5 it is recommended to use revised MSF proposed by Idriss as the lower bound of MSF defined as:

$$MSF = 10^{2.24}/M_w^{2.56} \tag{25}$$

As the upper bound, MSF is defined by the formula proposed by Andrus and Stokoe:

$$MSF = (M_w/7.5)^{-2.56} \tag{26}$$

For Mw >7.5, it is recommended to use revised MSF proposed by Idriss Equation (26).

3. Methodology

This research was carried out through a series of tests both directly in the field and in the laboratory to obtain the soil parameters needed as data to evaluate the liquefaction potential of a soil layer. Field testing is carried out in Sanur area, southeast of Denpasar City, Bali, with a distance of about 1 km from the beach. Groundwater level at the test site is about 1.5 meters below ground level. The tests included 6 pairs of CPT and SPT tests with very close distances for each pair, in order to make the test results can confirm each other. Laboratory testing was carried out at Soil Mechanics Laboratory of Bali State Polytechnic, including testing of grain size and unit weight. To evaluate soil resistance to liquefaction, in this study we will use a simplified procedure method described in detail in Report from the 1996 NCEER and 1998 NCEER / NSF Workshop on Evaluation of Liquefaction Resistance of Soils [10].

4. Results and discussion

The results of the CPT and SPT tests at each test point are presented in Figure 1. The SPT test is conducted at intervals of 2 meters up to a depth of 6 meters. The N_m value between the depths reviewed, is considered to be linier and determined by means of interpolation. The SPT test results show a small N_m value, less than 10, at all test points. According to Halim Asmar [1] the soil layer with a value of Nm <25 is suspectable to experience liquefaction. The CPT test results show that up to a depth of 6 meters, the average qc_1 value is between 11.3 - 16.6 kg/cm^2, except for the S2 point of 21.9 kg/cm^2. Soil Behaviour Type Index I_c ranges from 1.6-2.6. Based on this Ic value it can be assumed that the soil layer at the test site is silty sand to sandy silt. Unit weight ranges from 1.617–1.837 gr/cm^3 with finest content ranges from 6 – 9%. The evaluation of liquefaction potential based on CPT test showed that at earthquake magnitude Mw = 4.0, the soil at the test site did not experience liquefaction. At Mw = 5.0 most of the soil layer is liquefied. At Mw = 6.0, almost all of soil layers experience liquefaction, except at point 2 at a depth of 4 meters. The SF calculations based on the CPT test at Point 1 for Mw = 6 are presented in Table 1. The graph of SF against liquefaction for various earthquake magnitudes is shown in Figure 2.
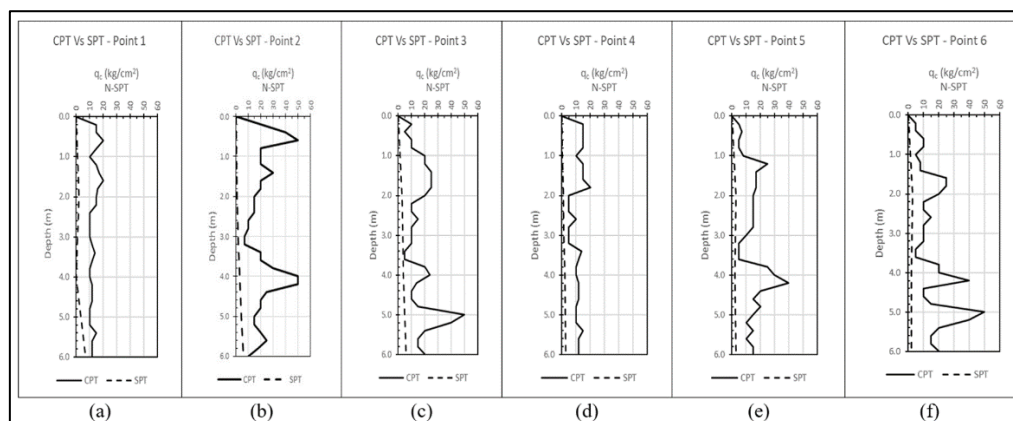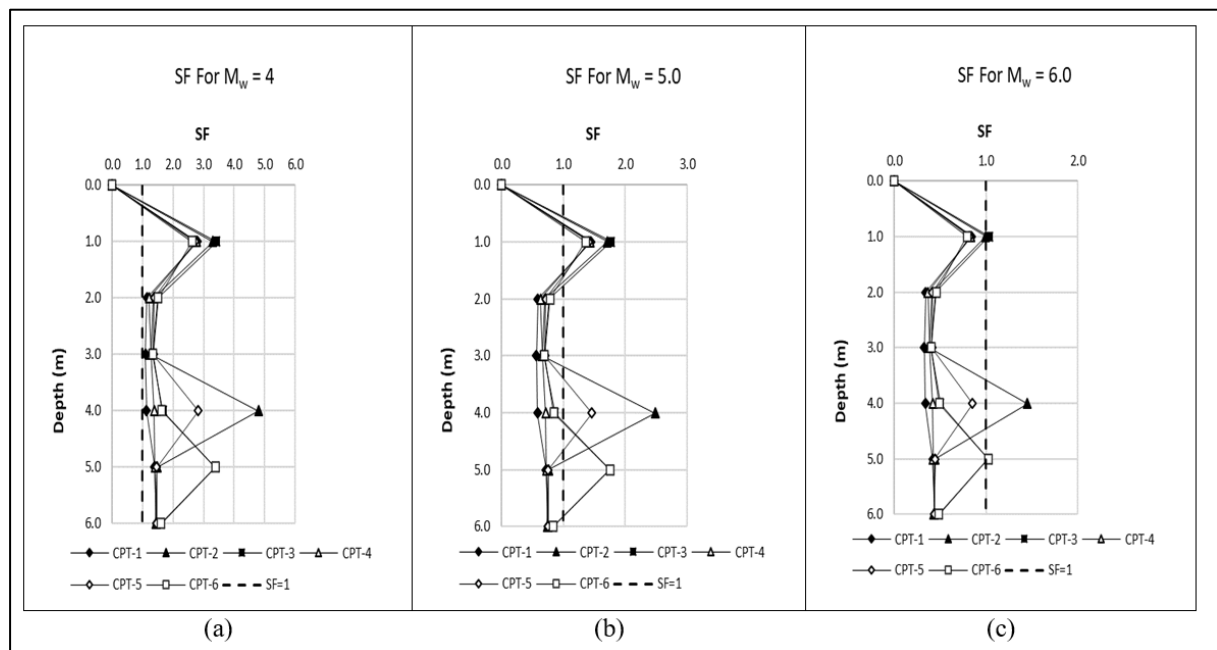

Figure 1. Soil test result – CPT and SPT at the appropriate points: (a) Point 1, (b) Point 2, (c) Point 3, (d) Point 4, (e) Point 5 and (f) Point 6.

Table 1. SF against liquefaction based on CPT-1 with $M_w = 6$.

D	q_{c1}	Tw	f_s	γ	σ_{vo}	σ'_{vo}	F	Q	Ic	C_Q	q_{c1N}	K_c	$(q_{c1N})_{cs}$	r_d	a_{max}	CSR	$CRR_{7,5}$	MSF	SF	Criteria
m	kg/cm²	kg/cm²		gr/cm³	kg/cm²	kg/cm²	%													
1.00	10.0	13.0	0.30	1.703	0.17	0.17	3.1	23.6	2.1	1.7	16.7	1.5	24.8	0.99	0.25	0.16	0.07	1.93	0.84	Liquefaction
2.00	15.0	20.0	0.33	1.617	0.323	0.123	2.3	41.4	1.9	1.7	25.0	1.2	29.7	0.99	0.25	0.42	0.07	1.93	0.34	Liquefaction
3.00	10.0	14.0	0.40	1.617	0.485	0.185	4.2	21.9	2.1	1.7	16.7	1.5	25.4	0.98	0.25	0.42	0.07	1.93	0.33	Liquefaction
4.00	10.0	15.0	0.50	1.617	0.647	0.247	5.3	18.6	2.2	1.7	16.7	1.7	27.8	0.97	0.25	0.41	0.07	1.93	0.34	Liquefaction
5.00	10.0	15.0	0.50	1.819	0.91	0.41	5.5	14.1	2.3	1.6	15.5	2.0	31.3	0.97	0.25	0.35	0.08	1.93	0.42	Liquefaction
6.00	12.0	15.0	0.25	1.819	1.091	0.491	2.3	15.4	2.3	1.4	16.9	2.0	34.2	0.96	0.25	0.35	0.08	1.93	0.44	Liquefaction

Figure 2. Safety factor SF for any earthquake magnitude based on CPT: (a) Mw = 4, (b) Mw = 5, (c) Mw = 6.

The calculation of SF based on SPT test at point B1 with earthquake magnitude Mw = 6 are presented in Table 2. The graphs of SF against liquefaction for various M_w based on the SPT test are given in Figure 3. Based on the SPT test, at $M_w = 4$ soil layers do not experience liquefaction. At $M_w = 5$ most of the soil layers undergo liquefaction and at $M_w = 6$, all of soil layers undergo liquefaction. It can be seen that the CPT and SPT tests provide evaluation results of potential liquefaction that are not much different. The comparison of SF between CPT and SPT for $M_w = 6$ is presented in Figure 4. It can be seen that the SF rate between CPT and SPT for the corresponding depth gives results that are not much different.

Table 2. SF against liquefaction based on SPT-1 with Mw = 6.

D	N_m	γ	σ_{vo}	σ'_{vo}	C_N	$(N_1)_{60}$	α	β	$(N_1)_{60cs}$	r_d	a_{max}	CSR	MSF	$CRR_{7,5}$	SF	Criteria
m		kN/m³	kN/m²	kN/m²												
1	1.0	17.03	17.03	17.03	2.42	1.7	0.03	1.00	1.8	0.99	0.25	0.16	1.93	0.10	0.62	Liquefaction
2	2.0	17.03	34.06	29.16	1.85	2.6	0.03	1.00	2.7	0.99	0.25	0.19	1.93	0.11	0.58	Liquefaction
3	1.0	16.20	48.60	33.90	1.72	1.2	0.03	1.00	1.3	0.98	0.25	0.23	1.93	0.10	0.42	Liquefaction
4	1.0	16.20	64.80	40.30	1.58	1.1	0.03	1.00	1.2	0.97	0.25	0.25	1.93	0.10	0.38	Liquefaction
5	3.5	18.20	91.00	56.70	1.33	3.3	0.03	1.00	3.4	0.97	0.25	0.25	1.93	0.12	0.46	Liquefaction
6	7.0	18.20	109.20	65.10	1.24	6.2	0.03	1.00	6.2	0.96	0.25	0.26	1.93	0.16	0.60	Liquefaction

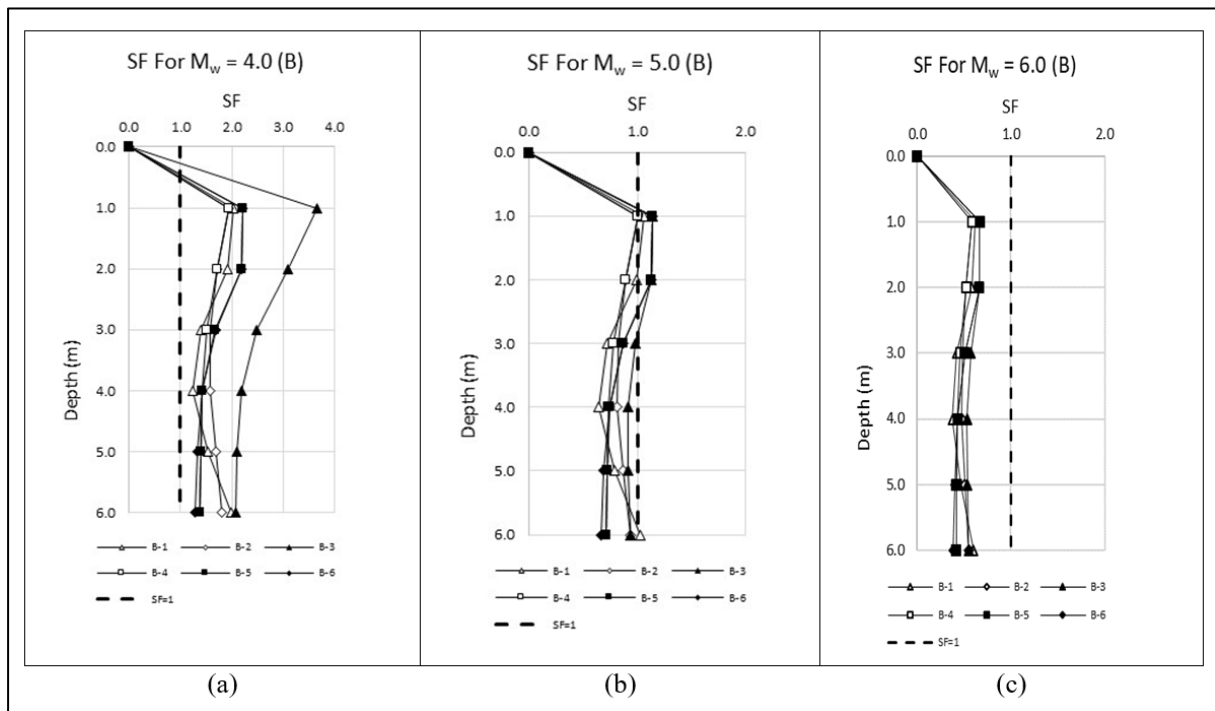**Figure 3.** Safety factor SF for any earthquake magnitude based on SPT: (a) Mw = 4, (b) Mw = 5, (c) Mw = 6.
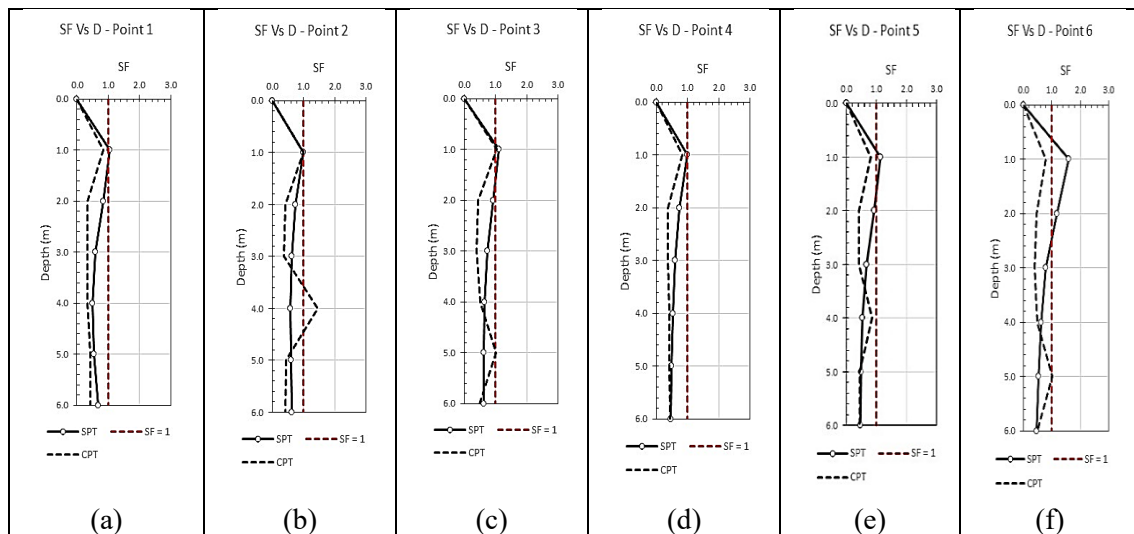

Figure 4. Comparison SF between CPT and SPT for Mw = 6: (a) Point 1, (b) Point 2, (c) Point 3, (d) Point 4, (e) Point 5 and (f) Point 6.

5. Conclusions

Based on the results of this evaluation, it can be concluded that the soil layer at the test site has the potential to experience liquefaction for earthquake magnitude > 5, using both CPT and SPT data. Analysis of potential liquefaction using CPT and SPT data gives results that are not much different.

6. References

[1]   Hasmar H A H 2013 *Dinamika Tanah Dan Rekayasa Kegempaan* (Yogyakarta: UII Press)

[2]   Idriss I M and Boulanger R W 2008 *Soil Liquefaction During Earthquakes* (Oakland: EERI MNO-12)

[3]   Orense R P, Towata I and Chouw N 2014 *Soil Liquefaction During Recent Large-Scale Earthquakes* (London: Taylor & Francis Group)

[4]   Tijow K C, Sompie O B A and Ticoh J H 2018 *Jurnal Sipil Statik* **6** 491

[5]   Hannah L and Veen V T 2015 *CPT Prediction of Soil Behaviour Type Liquefaction Potential and Ground Settlement in North-West Christchurch* (New Zealand: University of Canterbury)

[6]   Naghizaderokni M, Hassanlo M and Ramzani M 2017 *International Journal of GEOMATE* **12** 59-65

[7]   Misliniyati R, Mawardi, Besperi, Razali M R and Muktadir R 2013 *Jurnal Inersia* **5** 69-75

[8]   Kumar V, Venkatesh K and Kumar Y 2012 *Journal of Structural and Civil Engineering* **1** 35-53

[9]   Prawirodikromo W 2012 Seismologi *Teknik dan Rekayasa Kegempaan* (Yogyakarta: Pustaka Pelajar)

[10]   Youd T L, Idris I M, Andrus R D, Arango I, Castro G, Christian J T, Dobri R, Finn L, Harder Jr L F, Koester J P, Liao S S C and Marcuson W F 2001 *Journal of Geotechnical and Geoenvironmental Engineering* **127** 817-833

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