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Assessment of liquefaction potential of sand distributed in the coastal area of Ninh Thuan based on SPT values

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ABSTRACT

The liquefaction potential of sandy soil has been widely investigated in the world for many years, especially after the Niigata and Alaska earthquakes in 1964. In which, the SPT values were mostly used for the evaluation of liquefaction potential. In Vietnam, the potential for soil liquefaction has been recently investigated. In the coastal area of Ninh Thuan province, the sand is widely distributed and often exposed on the surface. Additionally, many wind power farms have been built in this area. Thus, it is necessary to evaluate the liquefaction potential of sand distributed in this area. In this study, the potential for sand liquefaction in the coastal area of Ninh Thuan will be evaluated based on SPT values. Two common methods for evaluation of liquefaction potential proposed by Seed and Idriss; Idriss and Boulanger were employed. The research results show that at peak ground acceleration (a_{max}) = 0.07 g and M = 5.5, the sand in the study area is non-liquefiable (safety factor against liquefaction, FS $\geq$ 1.48). However, at a_{max} = 0.18 g and M = 7.5, the FS varies from 0.78 to 3.66 for the Seed and Idriss method and from 0.68 to 2.05 for the Idriss and Boulanger method. In general, the FS obtained from the Seed and Idriss method is higher than that obtained from Idriss and Boulanger method. Nevertheless, this insignificantly affects the overall assessment of liquefaction potential. At a_{max} = 0.18 g and M = 7.5, the sand distributed in the study area can be liquefied at a depth of 17 m. However, it is not liquefied when the Nspt is higher than 28 ((N1) 60cs > 20).

1. Introduction

Liquefaction is a phenomenon that often occurs in saturated sandy soil under dynamic or cyclic loading such as earthquake, machine operation, traffic load, wind turbine, sea wave, etc,

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and causes severe damage to the construction. Liquefaction is identified as one of the most serious threats of earthquakes. However, since the occurrence of liquefaction depends on many factors such as the characteristics of soil strata, the properties of the soil layer, and the depth of groundwater level, the assessment of liquefaction potential is a complex task in geotechnical engineering (Rezania et al., 2010). Therefore, the assessment of the potential for soil liquefaction has been received much attention from researchers around the world, especially after the 1964 Niigata and Alaska earthquakes (Lentini and Castelli, 2019; Longbir and Arjun, 2017).

In general, the soil liquefaction potential can be assessed directly based on laboratory tests such as cyclic triaxial test, resonant column, cyclic torsional shear (Lentini and Castelli, 2019), and indirectly based on the results of the in situ tests such as the Standard Penetration Test (SPT), Cone Penetration Test (CPT), shear velocity (Vs), Becker Penetration Test (BPT) (Youd and Idriss, 2001). In indirect methods, assessment of the potential for soil liquefaction based on SPT values is one of the most common methods because of the abundance of SPT values. The simplified method for the evaluation of the potential for soil liquefaction based on SPT values was first proposed by (Seed and Idriss, 1971). The simplified method of Seed and Idriss (1971) was then modified and improved by others (Bolton Seed et al., 1985; Cetin et al., 2004; Idriss and Boulanger, 2006; Seed et al., 1983)

In Vietnam, the potential for soil liquefaction has been widely investigated by some authors (Bui et al., 2014, 2016; Bui and Le, 2014; Nguyen and Le, 2014, 2016; Nguyen, 2020). However, these investigations are only based on laboratory testing to evaluate the liquefaction potential of sand. Recently, some investigations on soil liquefaction based on SPT values have been conducted. Nguyen and Bui (2020) investigated the liquefaction potential of sand distributed in Quang Tri province based on SPT values. In their study, the liquefaction potential is assessed through three parameters, including safety factor against liquefaction (FS), liquefaction potential index (LPI), and liquefaction severity number (LSN). The research results showed that the sand distributed in Quang Tri province can be liquefied under the earthquake magnitude of 7 Richter. Nu et al. (2021) examined the liquefaction potential for sand distributed in the north-central coast of Vietnam and indicated that the sand at the depth of 18 m and the $(N1)_{60cs}$ value of less than 20 has a high liquefaction potential (FS<1). In general, the sand distributed in different areas has different liquefaction potential because of the differences in sand density, groundwater level, earthquake magnitude, and peak ground acceleration.

In Vietnam, along with economic development, the demand for construction works in coastal areas is now increasing, especially wind power projects. Additionally, in the coastal areas of Vietnam, the sand is widely distributed and often exposed on the surface which is susceptible to liquefaction. Based on the cyclic triaxial test, Nguyen (2020) showed that the loose sand distributed at the depth varied from 2.5 to 11.5 m in Soc Trang province could be liquefied under wind turbine operation with the cyclic stress ratio (CSR) ≥ 0.16. Thus, it is necessary to evaluate the liquefaction potential of sand distributed in different coastal areas of Vietnam under cyclic or dynamic loads. With this purpose, in this study, the liquefaction potential of sand distributed in the coastal area of Ninh Thuan province where many wind power farms are under building will be assessed based on SPT values. Two methods for the evaluation of sand liquefaction potential, namely simplified and semi-empirical procedures were employed in this investigation. Based on the calculated results, a comparison between the two methods was also made.

2. Study area

The coastal plain of Ninh Thuan occupies 22.4% of the province's area. Along the coastal area of Ninh Thuan, there is a fault line of 109÷110 degrees. Every year, earthquakes with a magnitude from 4.7 to 5.2 Richter occur in this area ("https://vietnamnet.vn/vn/khoa-hoc/van-de-an-toan-dia-diem-nmdhn-ninh-thuan 204103.html,"). In Vietnam, there were some earthquakes with a magnitude of up to 6.7÷6.8 Richter that occurred in Dien Bien (1935) and Tuan Giao (1983) (Nguyen, 2008). Thus, some large construction works in this area such as wind power farms and nuclear power plants must be designed for safety against earthquakes with a magnitude of higher or equal to 7.5 Richter. The coastal area of Ninh Thuan is one of the areas with the most potential for wind power development in Vietnam. Currently, there are dozens of wind power projects that have been built in this area. In most of wind power farms, the sand deposits are exposed on the surface with a thickness of tens of meters (Nam Mien Trung Ltd. Co., 2020). The origin of sand sediment here is mainly formed by the wind, so the particles is fine and poorly graded. Thus, the sand is susceptible to liquefy under dynamic loads. In this study, the data from 11 boreholes (49 SPT points) along the Ninh Thuan coastal areas were collected to evaluate the liquefaction potential. The location of the boreholes is shown in Figure 1. The characteristics of the sand layer and groundwater level in 11 boreholes are listed in Table 1. As shown, the sand layer is distributed from the surface to a depth of 17.3 m. The SPT values vary from 5 to 38 blows, and the fine content (<0.075 mm) ranges from 14.5% to 50%.

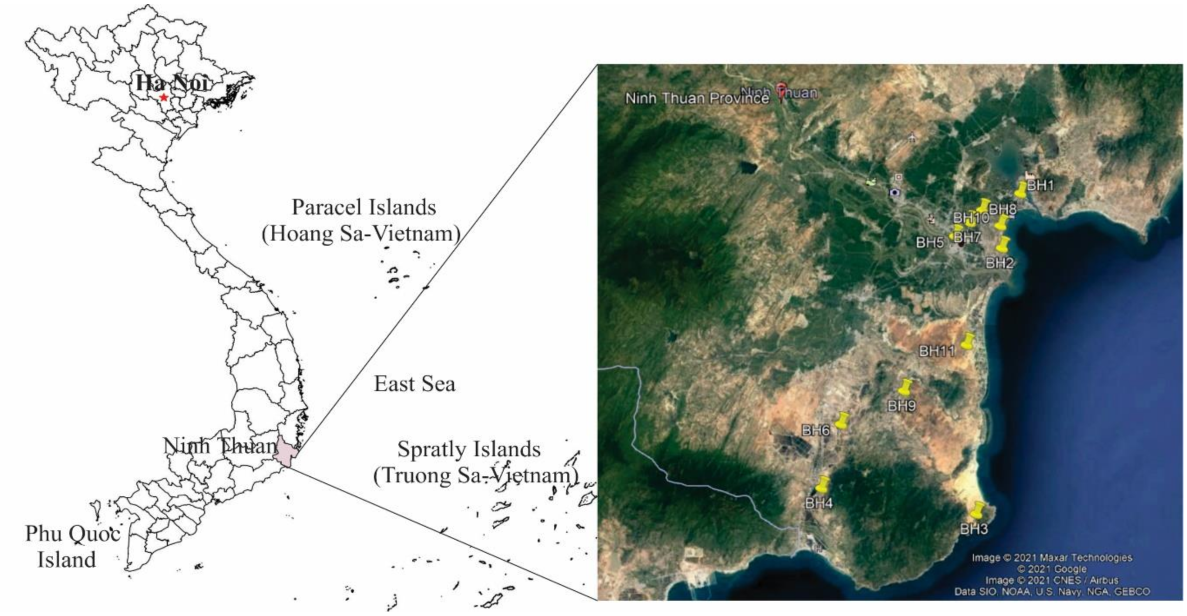


Figure 1. Study area and location of boreholes.

Table 1. Characteristics of the sand layer and groundwater level in 11 boreholes.

No.	Borehole	Sand layer (m)	Groundwater level (m)	Properties of the sand layer	
				SPT (Blows)	Fine content, FC (%)
1	BH1	0-12	7	8-29	23.0-26.0
2	BH2	0-8	8	6-26	20.0-23.0
3	BH3	0-3	None	16-17	24.0-27.2
4	BH4	0-9	None	10-24	14.5-17.1
5	BH5	0-5.5	6	5-9	20.5-23.1
6	BH6	0-14	None	21-38	46.0-50.0
7	BH7	0-8	3	13-20	23.1-24.7
8	BH8	0-8	6	8-16	22.6-24.2
9	BH9	0-6	6	6-19	19.7-21.6
10	BH10	0-8	10	10-17	18.1-19.4
11	BH11	0-17.3	12	14-28	20.8-23.1

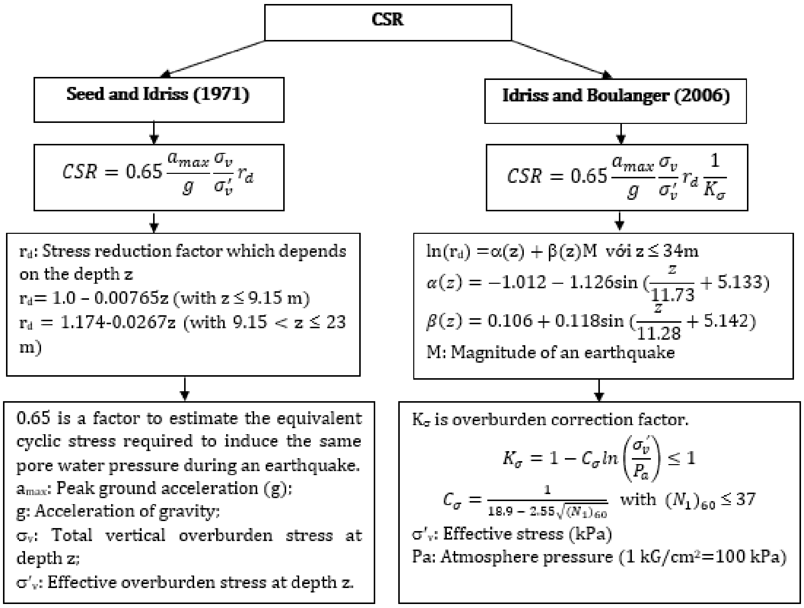
In the Ninh Thuan area, the peak ground acceleration (a_{max}) is about 0.056 g with an earthquake scale of VII (MSK-64) which equals the earthquake magnitude (M) of 5.5 Richter. With the Importance Factor of 1.25, the a_{max} is 0.07 g. In this coastal area, many wind power farms have been planned and some have been built such as Trung Nam, BIM, Adani, 7A, etc. These wind power farms are designed to be stable under the M of 7.5 Richter. The value of a_{max} generated in each earthquake depends on the magnitude of the earthquake, the geology structure, the depth of focus, and the frequency of ground motion. Some authors have proposed the correlation between earthquake magnitude or intensity and a_{max} for some areas in the world such as Tehran-Iran (Trifunac and Brady, 1975), Taiwan (Wu et al., 2003), Costa Rica (Linkimer, 2008). For M = 7.5 (Modified Mercalli Intensities of 7 to 8), the a_{max} can range from 0.18 g to 0.34 g (Wu et al., 2003). However, there is no correlation between earthquake magnitude and a_{max} in the Ninh Thuan area, so for M = 7.5 in this area, the a_{max} value is assumed to be equal to 0.18g. Therefore, in this study, the a_{max} of 0.07 g (M = 5.5) and 0.18 g (M = 7.5) were taken for the investigation.

3. Assessment of liquefaction potential based on SPT values

The SPT-based liquefaction potential can be evaluated based on different methods, such as Iwasaki et al. (1984); Seed and Idriss (1971); Tokimatsu and Yoshimi (1983); Idriss and Boulanger (2006). In this study, the simplified procedure proposed by Seed and Idriss (1971) and the semi-empirical procedure proposed by Idriss and Boulanger (2006) will be employed to evaluate the liquefaction potential. These methods are based on the calculation of the cyclic stress ratio (CSR) induced by the earthquake and cyclic resistance ratio (CRR). However, there are some differences between the two methods in the calculation of corrected factors, including stress reduction coefficient (r_d), magnitude scaling factor (MSF), the overburden correction factor for cyclic stress ratio (K_σ), and the overburden correction factor for penetration resistances (C_N) (Idriss and Boulanger, 2006).

3.1. Cyclic stress ratio (CSR) and cyclic resistance ratio (CRR)

Cyclic stress ratio (CSR) can be determined based on the peak ground acceleration (a_{max}) induced by a given earthquake and depends on the motion of a specific site. Whereas the cyclic resistance ratio (CRR) is the ability of the soil to resist the shear stresses induced by the earthquake. The calculation of CSR and CRR is summarised and shown in Figures 2 and 3.



$$CSR = 0.65\frac{a_{max}}{g}\frac{\sigma_v}{\sigma_v'}r_d$$

r_d: Stress reduction factor which depends on the depth z

$r_d = 1.0 - 0.00765z$ (with $z \le 9.15$ m)

$r_d = 1.174 - 0.0267z$ (with $9.15 < z \le 23$ m)

0.65 is a factor to estimate the equivalent cyclic stress required to induce the same pore water pressure during an earthquake.

a_{max}: Peak ground acceleration (g);

g: Acceleration of gravity;

σ_v: Total vertical overburden stress at depth z;

σ'_v: Effective overburden stress at depth z.

$$CSR = 0.65\frac{a_{max}}{g}\frac{\sigma_v}{\sigma_v'}r_d\frac{1}{K_\sigma}$$

$\ln(r_d) = \alpha(z) + \beta(z)M$ với $z \le 34$m

$\alpha(z) = -1.012 - 1.126\sin\left(\frac{z}{11.73} + 5.133\right)$

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

M: Magnitude of an earthquake

K_σ is overburden correction factor.

$$K_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma_v'}{P_a}\right) \le 1$$

$$C_\sigma = \frac{1}{18.9 - 2.55\sqrt{(N_1)_{60}}} \quad \text{with } (N_1)_{60} \le 37$$

σ'_v: Effective stress (kPa)

Pa: Atmosphere pressure (1 kG/cm^2=100 kPa)

Figure 2. Diagram for calculation of CSR.

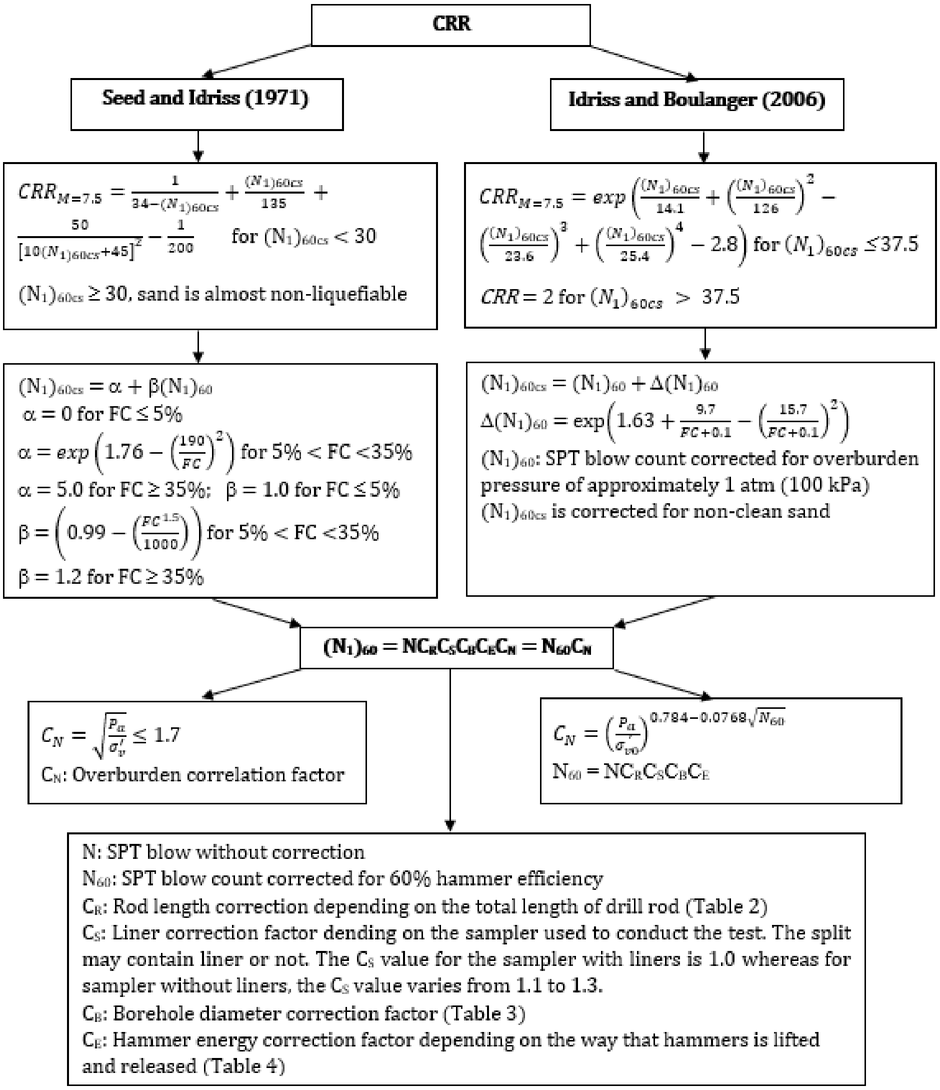


Figure 3. Diagram for calculation of CRR.

Table 2. Rod length correction (C_R) (Skempton, 1986).

Depth (m)	CR
<3	0.75
3-4	0.80
4-6	0.85
6-10	0.95
10-30	1.0

Table 3. Borehole diameter correction factor, CB.

Borehole diameter (mm)	CB
65 to 115 mm	1.00
150 mm	1.05
200 mm	1.15

Table 4. Hammer energy correction factor.

Hammer type	CE
Donut hammer	0.5-1.0
Safety hammer	0.7-1.2
Automatic-trip Donut type hammer	0.8-1.3

3.2. Factor of safety against liquefaction (FS)

The liquefaction potential is evaluated based on the factor of safety against liquefaction (FS) and can be determined as the following formula (Seed and Idriss, 1982):

$$FS = \frac{\mathrm{CRR}}{\mathrm{CSR}} = \frac{\mathrm{CRR}_{\mathrm{M=7.5}}}{\mathrm{CSR}}.\mathrm{MSF} \qquad (1)$$

Where MSF is the Magnitude scaling factor. MSF= $10^{2.24}/M^{2.56}$.

According to Idriss and Boulanger (2006), the factor of safety against liquefaction is calculated as follows:

$$FS = \frac{CRR}{CSR} = \frac{CRR_{M=7.5}}{CSR}.MSF.K_{\sigma} \qquad (2)$$

$$MSF = 6.9\exp\left(\frac{-M}{4}\right) - 0.058 \ \ (MSF \leq 1.8)$$

Based on the FS value, the liquefaction potential of soil can be evaluated and classified into three groups: Liquefiable, marginally liquefiable, and non-liquefiable (Table 5).

Table 5. Liquefaction potential of soil based on Fs.

Safety factor (FS)	Liquefaction potential
≤ 1	Liquefiable
$1 < FS \leq 1.2$	Marginally liquefiable
$FS > 1.2$	Non-liquefiable

4. Results and discussions

The results of calculated FS based on the methods of Seed and Idriss, Idriss and Boulanger as described above are shown in Tables 6 and 7 and their relationship with $(N_1)_{60cs}$ are plotted in Figures 4 and 5. As shown, the safety factor against liquefaction (FS) increases as the $(N_1)_{60cs}$ increases. In other words, the potential for liquefaction decreases with the increase of $(N_1)_{60cs}$. Additionally, it can be seen that the correlation between FS and $(N_1)_{60cs}$ based on the Seed and Idriss method (R^2 = 0.83) is stronger than that based on Idriss and Boulanger method (R^2=0.52). It means that the method of Seed and Idriss provides a stronger correlation between FS and $(N_1)_{60cs}$ rather than the method of Idriss and Boulanger.

As shown in Figure 4, the soil in the study area is not liquefied under the a_{max} of 0.07g, M = 5.5 with the FS ranging from 4.43 to 20.8 (Seed and Idriss method) and from 1.48 to 8.88 (Idriss and Boulanger method). This indicates that the earthquake with a_{max} = 0.07 g and M = 5.5 cannot cause soil liquefaction in the studied area. However, with the a_{max} of 0.18 g and M = 7.5, the soil can be liquefied (FS ≤ 1), marginally liquefied (1 < Fs ≤ 1.2), or non-liquefied (Fs >1.2) with the FS ranging from 0.78 to 3.66 (Seed and Idriss method) and from 0.68 to 2.05 (Idriss and Boulanger method). For both calculation methods, the sand in the study area is non-liquefiable when the $(N_1)_{60cs}$ is higher than 20 at the a_{max} of 0.18 g.

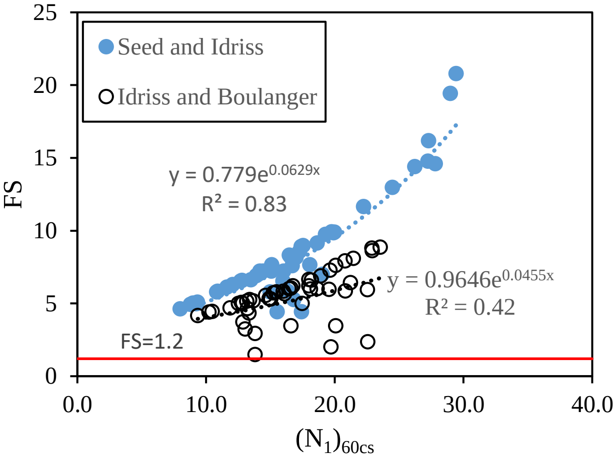


Figure 4. Relationship between FS and (N1)60cs (amax = 0.07 g, M = 5.5).

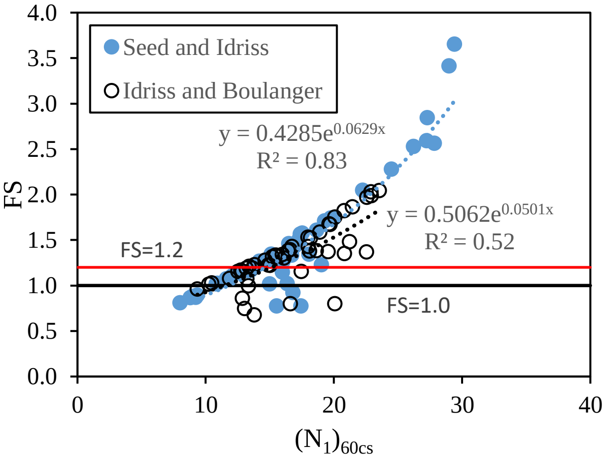


Figure 5. Relationship between FS and (N1)60cs (amax = 0.18 g, M = 7.5).

The pie chart comparing the results of FS at a_{max} of 0.18 g calculated by the Seed and Idriss method and Idriss and Boulanger method is shown in Figure 6. As shown, the percentage of liquefied points (Liq.) calculated by the Seed and Idriss method is 28.6%, slightly higher than that calculated by Idriss and Boulanger (22.4%). By contrast, the percentage of non-liquefied points (Non-Liq.) calculated by the Seed and Idriss method is 44.9%, which is slightly lower than that calculated by the Idriss and Boulanger method (49.0%). For marginally liquefiable (Mar-Liq.), the calculated results by the two methods are almost similar with a percentage of 26.5% from

Table 6. Calculated results of FS at PGA of 0.07g and M = 5.5.

Boreholes	Depth (m)	SPT (Blows)	Seed and Idriss		Evaluation	Idriss and Boulanger		Evaluation
			$(N_1)_{60cs}$	FS		$(N_1)_{60cs}$	FS	
BH1	1.0	8	10.9	5.85	Non-Liq.	12.7	5.07	Non-Liq.
	3.0	15	15.1	7.65	Non-Liq.	18.0	6.64	Non-Liq.
	5.0	26	19.7	9.92	Non-Liq.	22.9	8.81	Non-Liq.
	7.0	28	20.0	9.90	Non-Liq.	23.5	8.88	Non-Liq.
	9.0	27	18.0	7.66	Non-Liq.	21.2	6.43	Non-Liq.
	11.0	29	19.0	7.00	Non-Liq.	22.5	5.95	Non-Liq.
BH2	1.2	6	8.8	4.95	Non-Liq.	10.2	4.40	Non-Liq.
	3.2	11	12.7	6.53	Non-Liq.	15.4	5.78	Non-Liq.
	5.2	16	14.3	7.16	Non-Liq.	16.6	6.06	Non-Liq.
	7.2	26	19.9	9.86	Non-Liq.	22.9	8.64	Non-Liq.
BH3	1.3	16	17.5	8.98	Non-Liq.	20.8	7.92	Non-Liq.
	2.8	17	16.5	8.33	Non-Liq.	14.6	5.55	Non-Liq.
BH4	1.8	10	11.6	6.12	Non-Liq.	13.2	5.17	Non-Liq.
	3.8	25	19.3	9.75	Non-Liq.	21.4	8.09	Non-Liq.
	5.8	21	14.1	7.04	Non-Liq.	16.0	5.83	Non-Liq.
	7.8	24	13.5	6.63	Non-Liq.	15.0	5.31	Non-Liq.
BH5	1.0	5	8.0	4.62	Non-Liq.	9.4	4.18	Non-Liq.
	3.0	8	9.3	5.10	Non-Liq.	12.8	5.03	Non-Liq.
	5.0	9	9.2	4.97	Non-Liq.	11.9	4.69	Non-Liq.
BH6	1.0	21	29.4	20.8	Non-Liq.	20.1	7.61	Non-Liq.
	3.0	24	29.0	19.4	Non-Liq.	19.6	7.29	Non-Liq.
	5.0	27	27.3	16.2	Non-Liq.	18.2	6.61	Non-Liq.
	7.0	25	24.5	13.0	Non-Liq.	16.1	5.66	Non-Liq.
	9.0	31	26.2	14.4	Non-Liq.	18.1	6.00	Non-Liq.
	11.0	34	27.2	14.8	Non-Liq.	19.5	5.96	Non-Liq.
	13.0	38	27.8	14.6	Non-Liq.	20.8	5.86	Non-Liq.
BH7	1.0	13	15.0	5.80	Non-Liq.	17.5	5.01	Non-Liq.
	3.0	15	15.1	7.21	Non-Liq.	18.0	6.21	Non-Liq.
	5.0	18	16.0	7.22	Non-Liq.	18.6	6.01	Non-Liq.
	7.0	20	17.4	4.43	Non-Liq.	20.1	3.48	Non-Liq.
BH8	1.0	8	10.8	5.80	Non-Liq.	12.5	5.01	Non-Liq.
	3.0	13	14.2	7.21	Non-Liq.	16.7	6.21	Non-Liq.
	5.0	16	14.4	7.22	Non-Liq.	16.4	6.01	Non-Liq.
	7.0	14	15.5	4.43	Non-Liq.	16.6	3.48	Non-Liq.
BH9	1.0	6	9.0	5.02	Non-Liq.	10.5	4.46	Non-Liq.
	3.0	11	12.8	6.58	Non-Liq.	15.5	5.80	Non-Liq.
	5.0	19	16.5	8.22	Non-Liq.	18.9	6.89	Non-Liq.
BH10	2.0	10	12.1	6.30	Non-Liq.	13.8	1.48	Non-Liq.
	4.0	15	14.2	7.18	Non-Liq.	19.7	2.01	Non-Liq.
	6.0	17	13.9	6.92	Non-Liq.	22.6	2.37	Non-Liq.
BH11	1.0	14	17.4	8.92	Non-Liq.	13.4	5.26	Non-Liq.
	3.0	18	22.2	11.7	Non-Liq.	15.2	5.72	Non-Liq.
	5.0	20	17.3	8.59	Non-Liq.	13.6	5.17	Non-Liq.
	7.0	23	18.6	9.16	Non-Liq.	14.9	5.32	Non-Liq.
	9.0	23	17.0	8.18	Non-Liq.	13.2	4.66	Non-Liq.
	11.0	24	16.7	7.60	Non-Liq.	13.3	4.34	Non-Liq.
	13.0	25	16.0	6.55	Non-Liq.	12.9	2.21	Non-Liq.
	15.0	26	16.3	5.83	Non-Liq.	13.0	1.93	Non-Liq.
	17.0	28	16.8	5.26	Non-Liq.	13.8	1.75	Non-Liq.

Table 7. Calculated results of FS at PGA of 0.18g and M = 7.5.

Boreholes	Depth (m)	SPT (Blows)	Seed and Idriss		Evaluation	Idriss and Boulanger		Evaluation
			$(N_1)_{60cs}$	FS		$(N_1)_{60cs}$	FS	
BH1	1.0	8	10.9	0.93	Liq.	12.7	1.05	Mar-Liq.
	3.0	15	15.1	1.21	Non-Liq.	18.0	1.38	Non-Liq.
	5.0	26	19.7	1.57	Non-Liq.	22.9	1.83	Non-Liq.
	7.0	28	20.0	1.57	Non-Liq.	23.5	1.84	Non-Liq.
	9.0	27	18.0	1.21	Non-Liq.	21.2	1.33	Non-Liq.
	11.0	29	19.0	1.11	Mar-Liq.	22.5	1.24	Non-Liq.
BH2	1.2	6	8.8	0.78	Liq.	10.2	0.91	Liq.
	3.2	11	12.7	1.03	Mar-Liq.	15.4	1.20	Mar-Liq.
	5.2	16	14.3	1.13	Mar-Liq.	16.6	1.26	Non-Liq.
	7.2	26	19.9	1.56	Non-Liq.	22.9	1.79	Non-Liq.
BH3	1.3	16	17.5	1.42	Non-Liq.	20.8	1.64	Non-Liq.
	2.8	17	16.5	1.32	Non-Liq.	14.6	1.15	Mar-Liq.
BH4	1.8	10	11.6	0.97	Mar-Liq.	13.2	1.07	Mar-Liq.
	3.8	25	19.3	1.54	Non-Liq.	21.4	1.68	Non-Liq.
	5.8	21	14.1	1.11	Mar-Liq.	16.0	1.21	Non-Liq.
	7.8	24	13.5	1.05	Mar-Liq.	15.0	1.10	Mar-Liq.
BH5	1.0	5	8.0	0.73	Liq.	9.4	0.87	Liq.
	3.0	8	9.3	0.81	Liq.	12.8	1.04	Liq.
	5.0	9	9.2	0.79	Liq.	11.9	0.97	Liq.
BH6	1.0	21	29.4	3.29	Non-Liq.	20.1	1.58	Non-Liq.
	3.0	24	29.0	3.08	Non-Liq.	19.6	1.51	Non-Liq.
	5.0	27	27.3	2.56	Non-Liq.	18.2	1.37	Non-Liq.
	7.0	25	24.5	2.05	Non-Liq.	16.1	1.18	Mar-Liq.
	9.0	31	26.2	2.28	Non-Liq.	18.1	1.25	Non-Liq.
	11.0	34	27.2	2.34	Non-Liq.	19.5	1.24	Non-Liq.
	13.0	38	27.8	2.31	Non-Liq.	20.8	1.22	Non-Liq.
BH7	1.0	13	15.0	0.92	Non-Liq.	17.5	1.04	Mar-Liq.
	3.0	15	15.1	1.14	Non-Liq.	18.0	1.29	Mar-Liq.
	5.0	18	16.0	1.14	Liq.	18.6	1.25	Non-Liq.
	7.0	20	17.4	0.70	Liq.	20.1	0.72	Liq.
BH8	1.0	8	10.8	0.92	Liq.	12.5	1.04	Liq.
	3.0	13	14.2	1.14	Mar-Liq.	16.7	1.29	Mar-Liq.
	5.0	16	14.4	1.14	Mar-Liq.	16.4	1.25	Mar-Liq.
	7.0	14	15.5	0.70	Liq.	16.6	0.72	Liq.
BH9	1.0	6	9.0	0.79	Liq.	10.5	0.93	Liq.
	3.0	11	12.8	1.04	Mar-Liq.	15.5	1.20	Mar-Liq.
	5.0	19	16.5	1.30	Non-Liq.	18.9	1.43	Non-Liq.
BH10	2.0	10	12.1	1.00	Mar-Liq.	13.8	1.11	Mar-Liq.
	4.0	15	14.2	1.14	Mar-Liq.	19.7	1.51	Non-Liq.
	6.0	17	13.9	1.09	Mar-Liq.	22.6	1.78	Non-Liq.
BH11	1.0	14	17.4	1.41	Non-Liq.	13.4	1.09	Mar-Liq.
	3.0	18	22.2	1.84	Non-Liq.	15.2	1.19	Mar-Liq.
	5.0	20	17.3	1.36	Non-Liq.	13.6	1.07	Mar-Liq.
	7.0	23	18.6	1.45	Non-Liq.	14.9	1.10	Mar-Liq.
	9.0	23	17.0	1.29	Non-Liq.	13.2	0.97	Liq.
	11.0	24	16.7	1.20	Non-Liq.	13.3	0.90	Liq.
	13.0	25	16.0	1.04	Mar-Liq.	12.9	0.77	Liq.
	15.0	26	16.3	0.92	Liq.	13.0	0.67	Liq.
	17.0	28	16.8	0.83	Liq.	13.8	0.76	Liq.

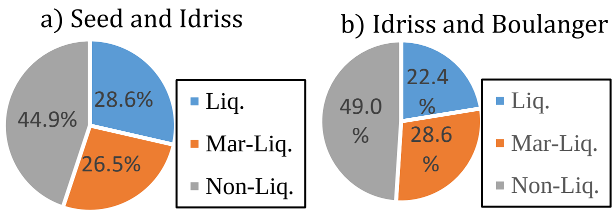


Figure 6. Pie chart comparing the results of FS calculated by different methods (a_{max} = 0.18 g).

the Seed and Idriss method and 28.6% from the Idriss and Boulanger method. In general, the differences in FS obtained from the two calculation methods are not significant.

The relationships between FS, depth, and SPT values are presented in Figure 7. In general, as shown in Figures 5, and 7, the calculated FS values based on the method of Seed and Idriss are higher than those based on Idriss and Boulanger method, especially at high FS values (FS > 2). This is due to the difference in the calculation of some correction factors between the two methods. However, this does not affect the overall assessment of soil liquefaction. For both calculation methods, Figure 7a shows that the sand distributed at the depth of 17 m (BH11) can be liquefied. This is affected by the groundwater level in this borehole. In Vietnam, based on the cyclic shear test, Bui et al. (2014) showed that the sand distributed in the Hanoi area beyond depths of 18 m was not liquefied at the frequency of 2 Hz and cyclic stress amplitude of 50 kPa. Based on SPT values, Nu et al. (2021) concluded that the sand on the north-central coast of Vietnam could be liquefied at a depth of up to 18 m. In the world, as reported in the literature, sand is almost no liquefaction at depths higher than 30 m (Dobry et al., 1982; Seed and Idriss, 1971). According to Florin and Ivanov (1961), the sand is not liquefied beyond depths of 15 m, even for very loose sand. In the laboratory, at high confining pressures of 7 to 70 kG/cm^2 (equal to 30 to 300 m), the liquefaction potential was found at such depths (Bishop, 1965). Thus, in the study area, the sand at a depth of higher than 17 m can be liquefied. Regarding SPT values, as shown in Figure 7b, the sand is non-liquefiable when the SPT values are higher than 28 blows ((N1)60cs > 20). Similarly, Nu et al (2021) also indicated that the sand distributed on the north-central coast of Vietnam was non-liquefiable when the (N1)60cs values were higher than 20.

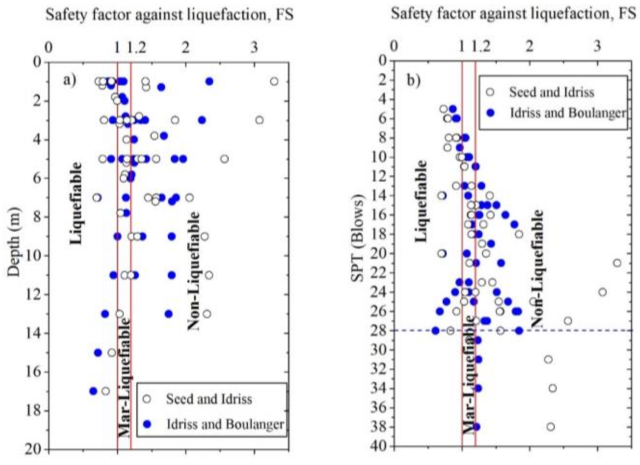


Figure 7. Relationship between FS, depth and SPT (a_{max} = 0.18g, M = 7.5).

5. Conclusions

In this study, the liquefaction potential of sand distributed in 11 boreholes in the coastal area of Ninh Thuan province has been evaluated based on SPT values at a_{max} of 0.07 g and 0.18 g. Two calculation methods, namely simplified and semi-empirical procedures, were employed to evaluate the potential for sand liquefaction. Some conclusions can be drawn as follows:

Although the method of Seed and Idriss can give the FS values higher than the method of Idriss and Boulanger, especially at high FS values, it insignificantly affects the overall assessment of soil liquefaction. In addition, the correlation between FS and $(N1)_{60cs}$ based on the Seed and Idriss method is stronger than that based on Idriss and Boulanger method.

For both calculation methods, the earthquake with the a_{max} = 0.07 g and M = 5.5 cannot cause sand liquefaction in the study area. However, with the a_{max} of 0.18 g and M = 7.5, the sand distributed at the depth of 17 m can be liquefied. Additionally, under such a_{max} value, the sand is liquefiable when the SPT values are lower or equal to 28 blows ($(N1)_{60cs} \leq 20$).

Author contributions

Duong Thanh Nguyen - Proposed the idea, writing - original draft, writing - review & editing;

Son Tan Nguyen - Data curation; Son Tan Nguyen, Ha Ngoc Thi Pham, and Hai Huu Phung - Formal analysis.

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