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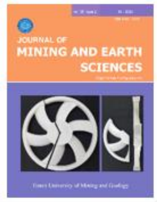

Determine attenuation coefficients of saturated sand from standard penetration test, applied to Nhon Hoi economic zone

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ABSTRACT

Vietnam is located in a strong earthquake zone with many areas having a ground acceleration (a_g) of greater than 0.08 g (TCVN 9386:2012). According to Vietnamese standard TCVN 9386:2012, the calculation and design of construction grade III or higher in areas where have ground acceleration $a_g \geq 0.08$ g must include seismic design. In calculations of earthquake-resistant pile foundation, the bearing capacity of the pile including the tip resistance strength (q_b) and endurance strength (f_i) need to be multiplied by attenuation coefficients (γ_{eq1} and γ_{eq2}). They are the most important parameters and depend on the soil types, saturation conditions, and earthquake intensity. The article introduces a method to determine the attenuation coefficients according to the standard TCVN 10304:2014. In addition, analyzing the theoretical basis to give the expression to determine attenuation coefficients based on the durability factor (I_{red}) and pore water pressure ratio (R_u) of saturated sand under the earthquakes. Furthermore, the paper presents a method to determine R_u from the results of standard penetration test (SPT) by combining the method of Seed and Alba (1986) with the method of Marcuson and Hynes (1990). In which, Seed and Alba's method was used to determine factor of safety against liquefaction (F_{SL}), and then Marcuson and Hynes' method was used to determine R_u from F_{SL}. The application in Nhon Hoi Economic Zone shows that: The silty fine grained sand, which is medium dense and saturated, has a attenuation coefficient of tip resistance $\gamma_{eq1} = 0.74 \div 0.76$ and attenuation coefficient of friction $\gamma_{eq2} = 0.90$; The fine grained sand, which is dense and saturated, has $\gamma_{eq1} = 0.79 \div 0.82$ and $\gamma_{eq2} = 0.94$.

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1. Introduction

Currently, the calculation of earthquake-resistant buildings in Vietnam is specified in the Vietnamese Standards TCVN 9386:2012 and TCVN 10304:2014. In particular, the standard TCVN 9386:2012 has regulations on cases that design for seismic resistance for buildings, based on two basic criteria: peak ground acceleration (ag) and the importance of the building. In regions with the peak ground acceleration ag ≥ 0.08 g, and construction grade III or higher must be designed for seismic resistance. Therefore, many types of buildings are required to seismic resistance design.

For works using pile foundations, the design of pile foundations in earthquake zones is specified in TCVN 10304:2014. Accordingly, the determination of the load-bearing capacity of the pile should be multiplied by attenuation coefficients of the working conditions of the ground. These coefficients can be obtained empirically or determined by testing piles and pile foundations subjected to simulated impact. The empirical determination (according to table 18 of this standard) has low reliability due to the wide division. The method of testing pile by simulated impact has high reliability, but is complicated and expensive. In fact, the calculation of the load-bearing capacity of piles in the earthquake zone in Vietnam is mainly based on the table of coefficients. In the literature, there are many studies on the deterioration of soil strength under earthquakes (Seed and Idriss, 1971; Seed and Alba, 1986; Marcuson and Hynes, 1990; Al-Karni, 2001), which can be applied to determine attenuation coefficients in conformity with this standard. However, there is currently no research of determination of attenuation coefficients in Vietnam.

Therefore, it is very important to synthesize and analyze the theoretical basis to give an expression to determine the attenuation coefficients. This allows updating new research results, and improving the reliability of the pile foundations design in earthquake zones. This article mainly focuses on researching for saturated sandy soil and proposes a method using standard penetration test (SPT) results, applied to saturated sand distributed in Nhon Hoi economic zone.

2. Theoretical basis and methodology

2.1. Deterioration in strength of saturated soil under dynamic load

Dynamic-induced loads cause a rapid increase in excess pore water pressure (ΔU) in saturated soil, that leads directly to reduction of soils strength. In practice, the pore water pressure ratio ($R_u = \Delta U/\sigma'$; where σ' is effective stress) is used to evaluate the influence of excess pore water pressure under dynamic loads. R_u depends on soil type, saturation, and stress conditions.

Al-Karni (2001) used the stability factor (I_{red}) to quantitatively evaluate the deterioration in strength according to ΔU:

$$I_{red} = \frac{\tau_{red}}{\tau} \tag{1}$$

With τ - the initial strength of the soil; τ_{red} - the strength of the soil under earthquakes.

By mathematical transformation from Coulomb's law, Al-Karni (2001) gave the expression to determine the durability factor:

$$I_{red} = \left[1 - \frac{1}{\left(1+\frac{C}{\sigma' tg\varphi}\right)}\frac{\Delta U}{\sigma'}\right] \tag{2}$$

For sandy soil, Nguyen et al. (2020) has established the expression to determine I_{red} and proposed the apparent friction angle (φ_{red}) to evaluate the durability of saturated sand under dynamic loads:

$$I_{red} = 1 - R \tag{3}$$

$$tg\varphi_{red} = I_{red}.tg\varphi \tag{4}$$

Thus, the degradation due to the increase in pore water pressure in the soil can be quantified knowing ΔU by using expressions (2), (3) depending on the soil type. However, dynamic load not only increases the pore water pressure, but also partially reduces the cohesive force of soil. Therefore, the evaluation of degradation in strength of cohesive soil under dynamic loads is quite complicated. The strength of sandy soil (loose soil) is determined by the friction force, so expression (4) can be used to determine the strength of sand for calculations of foundation under dynamic loads.

2.2. Attenuation coefficients and method of determination

According to TCVN 10304:2014, when designing pile foundations in high seismic harzard zones, the values of tip resistance strength (q_b) and endurance strength (f_i) need to be multiplied by attenuation coefficients for working conditions of foundation, respectively, γ_{eq1} and γ_{eq2}. These coefficients depend on soil type and earthquake intensity, which can be obtained from Table 1.

From the concept of attenuation coefficients, the following expression can be determined:

$$\gamma_{eq1} = \frac{q_{b(eq)}}{q_b} \quad (5)$$

$$\gamma_{eq2} = \frac{f_{i(eq)}}{f_i} \quad (6)$$

In which, q_b, f_i are the tip resistance strength and the endurance strength under normal conditions, respectively; $q_{b(eq)}$, $f_{i(eq)}$ are the tip resistance strength and the endurance strength under earthquake, respectively.

Theoretically, the tip resistance strength of sandy soil is determined as follows:

$$q_b = \sigma'.N_q \quad (7)$$

$$q_{b(eq)} = \sigma'.N_{q(eq)} \quad (8)$$

N_q and $N_{q(eq)}$ are respectively coefficients depending on the friction angle φ and φ_{red}, which can be looked up in a table or calculated by the expression:

$$N_q = e^{\pi tg\varphi} tg^2(45^o + \frac{\varphi}{2}) \quad (9)$$

$$N_{q(eq)} = e^{\pi tg\varphi_{red}} tg^2(45^o + \frac{\varphi_{red}}{2}) \quad (10)$$

Substitute (7), (8) into (5) we get:

$$\gamma_{eq1} = \frac{N_{q(eq)}}{N_q} = \frac{e^{\pi tg\varphi_{red}} tg^2(45^o + \frac{\varphi_{red}}{2})}{e^{\pi tg\varphi} tg^2(45^o + \frac{\varphi}{2})} \quad (11)$$

Replace expression (3), (4) into expression (11) and transform, we get:

$$\gamma_{eq1} = e^{(I_{red}-1)\pi tg\varphi}.\frac{tg^2(45^o + \frac{\varphi_{red}}{2})}{tg^2(45^o + \frac{\varphi}{2})} \quad (12)$$

The endurance strength for sandy soil is determined according to the expressions:

$$f_i = \sigma'_i.tg\varphi \quad (13)$$

$$f_{i(eq)} = \sigma'_i.tg\varphi_{red} \quad (14)$$

Therefore,

$$\gamma_{eq2} = \frac{f_{i(eq)}}{f_i} = \frac{tg\varphi_{red}}{tg\varphi} = I_{red} \quad (15)$$

Thus, with water-saturated sandy soil, expressions (11) or (12), (15) can be used to determine the attenuation coefficients γ_{eq1} and γ_{eq2} from I_{red} and φ_{red}, when knowing pore water pressure ratio R_u.

Table 1. Working condition coefficients γ_{eq1} and γ_{eq2} (according to TCVN 10304:2014).

Earthquake scales for houses and buildings	Working condition coefficients γ_{eq1} to correct q_b						Working condition coefficients γ_{eq2} to correct f_i				
	Compacted sand		Medium compacted sand		Sticky soil corresponds to the viscosity index		Compacted and medium-compacted sand		Cohesive soil corresponds to the viscosity index		
	Slightly wet and wet	Saturat-ed	Slightly wet and wet	Saturat-ed	I_l<0	$0 \le i_l \le 0.5$	Slightly wet and wet	Saturat-ed	I_L< 0	$0 \le I_L < 0.75$	$0.75 \le I_L < 1.0$
7	1	0.9	0.95	0.8	1	0.95	0.95	0.9	0.95	0.85	0.75
	0.9	0.5	0.85	0.4	1	0.9	0.85	0.5	0.9	0.8	0.75
8	0.9	0.8	0.85	0.7	0.95	0.9	0.85	0.8	0.9	0.8	0.7
	0.8	0.4	0.75	0.35	0.95	0.8	0.75	0.4	0.8	0.7	0.65
9	0.8	0.7	0.75	-	0.90	0.85	0.75	0.7	0.85	0.7	0.6
	0.7	0.35	0.60	-	0.85	0.7	0.65	0.35	0.65	0.6	-

Note: γ_{eq1}, and γ_{eq2} above are for driven piles, value of γ_{eq1}, and γ_{eq2} below are for bored pile.

2.3. The method of determining Ru from the results of the SPT

The pore water pressure ratio R_u depends on the soil type, stress conditions and dynamic load characteristics. Currently, there are many methods to determine R_u such as direct experimental methods (cyclic triaxial tests, cyclic direct shear tests) or methods of determination by experimental relationship. One of these methods is the estimation of R_u according to the factor of safety against liquefaction (F_{SL}) (Marcuson and Hynes, 1990).

The method of estimation R_u based on F_{SL} is widely used because of its convenience. The liquefaction ability of saturated sand is evaluated by F_{SL} coefficient, when $F_{SL} \leq 1$, the soil is liquefied; and $F_{SL} > 1$ means the soil is non-liquefied. The case of $F_{SL} > 1$ occurs when the stress conditions, strength and duration of the dynamic load are not enough to cause liquefaction, but still generate excess pore water pressure in the soil. According to Marcuson and Hynes (1990), R_u can be estimated according to FSL as shown in Figure 1. Accordingly, when $F_{SL} < 1.3$, R_u significantly increase and reachs the maximum value of 1 when $F_{SL} = 1$ (liquid state threshold). The larger the F_{SL}, the smaller R_u.

Thus, in order to determine R_u, it is necessary to determine the F_{SL}. The F_{SL} factor is the ratio of Cyclic Resistance Ratio (CRR) to Cyclic Stress Ratio (CSR):

$$F_{SL} = \frac{CRR}{CSR} \tag{16}$$

CRR depends on sand type, fine grain content and sand density, which can be determined by experimental relations with standard penetration test (SPT), with measurement of pore water pressure (CPTu) or by direct experiment. Based on the SPT value corresponding to 60% of the blows's energy $(N_1)60$, Seed and Alba (1986) built a chart that allows to determine the CRR corresponding to the earthquake intensity M = 7.5 for clean sand and dusty sand (according to fine particle content, FC) as shown in Figure 2. $(N_1)_{60}$ is determined by the following expression:

$$(N_1)_{60} = N.C_E.\ 9.79.\left(\frac{1}{\sigma'_{vo}}\right)^{0,5} \tag{17}$$

Where, N - the actual measured SPT value; σ'_{vo} (kPa) - the effective overburden stress; C_E - he effective energy factor, which can be taken as 0.5÷0.9.

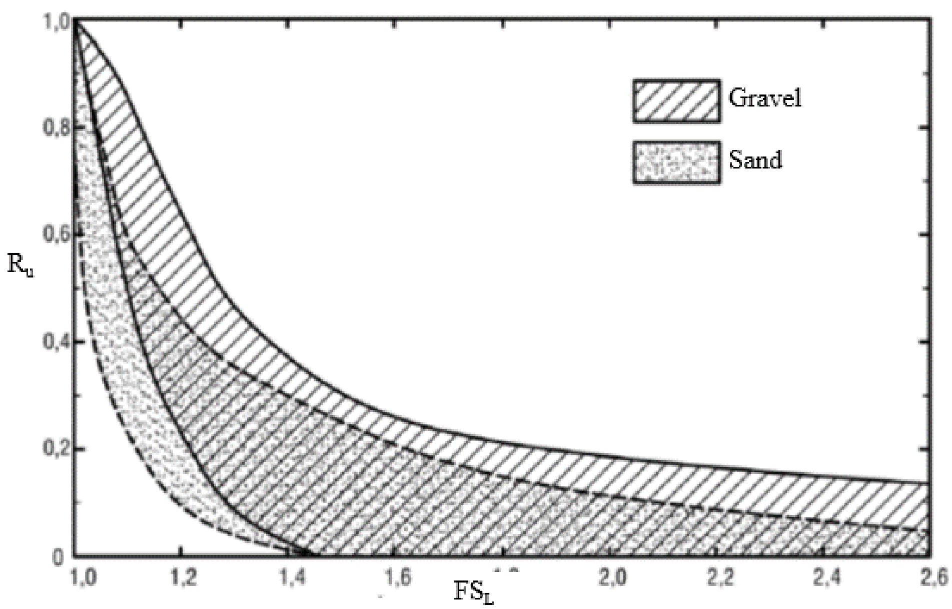

Figure 1. Relationship between F_{SL} and R_u of sandy soil and gravel (Marcuson and Hynes, 1990).

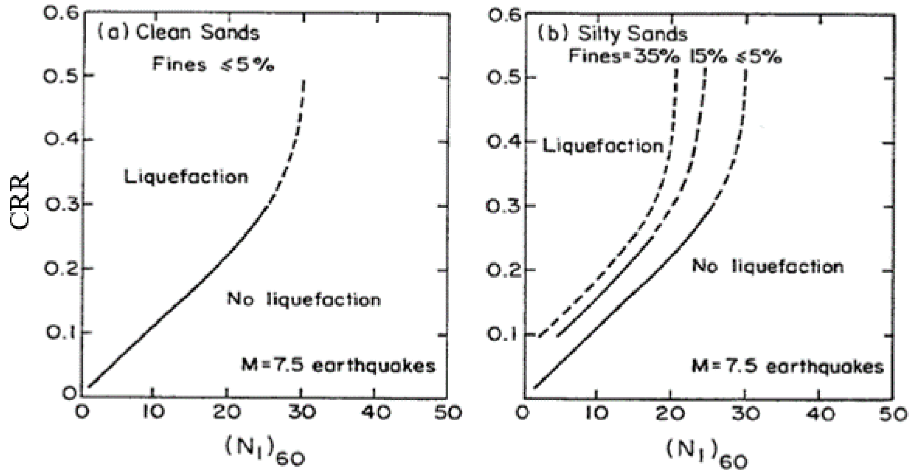

Figure 2. Relationship between CRR and $(N_1)_{60}$ (Seed and Alba, 1986).

CSR is the ratio of the dynamic shear stress caused by the earthquake to the effective overburden stress, as determined by Seed and Idriss (1971):

$$CSR = 0{,}65\left(\frac{a_g}{g}\right) r_d \left(\frac{\sigma_{vo}}{\sigma'_{vo}}\right) \qquad (18)$$

Where: a_g - the peak ground acceleration determined according to the seismic sub-partition table; r_d - the adjustment factor for depth, r_d = 1-0.012z, where z is the depth (m); σ_{vo} - the total vertical overburden stress.

Thus, the results of the SPT experiment can be used to determine the CRR according to the chart in Figure 2, combining with the expressions (16) and (18) to determine the F_{SL}, thereby determine R_u. The R_u value is used to determine the stability factor I_{red} and the apparent friction angle φ_{red}, thereby determining the attenuation coefficients according to expressions (11) and (15).

3. Research results and discussion

3.1. Features of construction planning and earthquakes in Nhon Hoi economic zone

Nhon Hoi economic zone is planned to 2040 with an area of 12,000 ha in Quy Nhon city and a part of Tuy Phuoc and Phu Cat districts in Binh Dinh province. This is a general economic zone, including: an industrial park, a seaport and seaport service area, a tourist area, a service and a new urban area and a renewable energy development zone. In which, there are many project items of medium to large scale (Figure 3).

Figure 3. Nhon Hoi economic zone.

According to the map of the ground acceleration zoning of Vietnam (TCVN 9386:2012), this area has a peak ground acceleration a_g changed from 0.0941 g to 0.1070 g, coresponding to a magnitude 7 earthquake in the MSK-64 scale. On the other hand, the coast in this studied area has shallow groundwater. The soil here is mainly of fine sand and about 10 meters thick. These are unfavorable features for the stability of the structure when subjected to earthquakes.

3.2. Typical stratigraphic features of Nhon Hoi economic zone

Based on the Geological-Mineral Map of Binh Dinh Province (Central Geological Federation, 2006), it can be seen that the planning area of construction works in Nhon Hoi economic zone belongs to the topography accumulated by marine-windy sediments mvQ_2^3 (Figure 4).

According to engineering geological survey documents in some projects, the stratigraphy in the area is mainly fine sand. At the study site, the typical stratigraphy consists of three soil layers: Layer 1 - Medium dense fine - silty Sand, yellow in color; Layer 2 - Dense fine Sand, yellowish brown in color; Layer 3 - Stiff, yellowish grey sandy Clay. Stratigraphic characteristics and sedimentary facies are summarized in Table 2. Groundwater in the studied area is mainly distributed in layer 2. The depth of water level is varied from 3.0 to 7.5 m. The part above water level is saturated with capillary water, from a depth of about 3.0÷4.0 m.

With the regional stratigraphic characteristics, the solution of friction pile foundation placed in layer 2 or layer 3 is possible for medium to large scale projects in Nhon Hoi economic zone.

3.3. Determine attenuation coefficients by empirical method

Figure 4. Characteristics of geological stratigraphic distribution of Nhon Hoi economic zone (According to the geological and mineral map of Binh Dinh province, 2006).

Table 2. Characteristic of stratigraphy of the study area.

Layer	Depth (m)		Average thickness (m)	Formation	Type of soil	SPT value/average value (blows)
	Surface	Bottom				
1	0	4.5-10.0	6.5	mvQ_2^{2-3}	Medium dense fine -silty Sand, yellow in color. FC =8.6%	(12÷26)/16
2	4.5-10.0	38.0-43.5	32.0	m, amQ_2^{2-3}	Dense fine Sand, yellowish brown in color. FC =6.4%	(31÷51)/43
3	38.0-43.5	50	11.5	amQ_1^3	Stift, yellowish grey sandy Clay	21÷50

Applying empirical method according to the standard TCVN 10304:2014 as presented in Section 2.2, attenuation coefficients of sandy soil in the study area were determined as shown in Table 3.

The attenuation coefficients of sand used for bored piles is significantly less than that of piles lowered by driving or pressing. The reason is the compaction of the sand around the pile which is significantly reduced during the drilling process to create bored pile holes. It is in contrast to the solution of driving or pressing piles.

3.4. Determination of attenuation coefficients from SPT results

On the basis of the method presented in Section 2, the determination of attenuation coefficients is applied to saturated sand with the density varied from medium (layer 1) to dense (layer 2). Using the results of the SPT (N) with specific parameters at the research site, the value $(N_1)_{60}$ was determined according to the expression (17). Then looking up the chart in Figure 3 with the fine grain content of each type of sand allows to determine the CRR. The CSR due to earthquake is calculated according to the expression (18) with the largest ground acceleration a_g = 0.1070 g. The FSL is then determined using expression (16). The summary of the calculation results is given in Table 4.

The factor of safety against liquefaction (F_{SL}) in layers 1 and 2 can be taken as 2.0 and 2.3, respectively. Using the relationships in Figure 2, the value of FS_L indicated that layer 1 has the largest R_u value of 0.10, meanwhile layer 2 has the largest R_u value of 0.06. Using these values of R_u, the durability factor I_{red} and φ_{red} for each type of soil can be determined according to expressions (3) and (4). In which, the internal friction angle of the sand layers is estimated according to the SPT value (based on TCVN 9351:2012). Attenuation coefficients are determined according to expressions (11) and (15). The summary of the calculation results is summarized in Table 5.

Table 3. Attenuation coefficients for sandy soil distributed in Nhon Hoi economic zone (for magnitude 7 earthquake).

Type of sand	Value of γ_{eq1} to correct q_b	Value of γ_{eq2} to correct f_i
Medium dense, wet fine -silty Sand (The sand above the capillary water level in Layer 1)	0.95	0.95
	0.85	0.85
Medium dense, saturated fine-silty Sand – (Layer 1)	0.80	0.90
	0.40	0.50
Dense, saturated fine Sand (Layer 2)	0.90	0.90
	0.50	0.50

Table 4. Results of determination FS_L from SPT.

Layer	z (m)	N	C_E	σ_{vo} (kPa)	U (kPa)	σ'_{vo} (kPa)	$(N_1)_{60}$	φ Độ	CRR	r_d	CSR	FS_L
Layer 1	4	15	0.70	78.0	9.8	68.2	14.9	29.7	0.16	0.952	0.08	2.00
	6	23	0.70	117.0	29.4	87.6	16.8	31.6	0.18	0.928	0.09	2.09
Layer 2	8	31	0.70	156.0	49.1	107.0	20.5	34.3	0.22	0.904	0.09	2.40
	10	33	0.70	196.0	68.7	127.3	20.0	34.9	0.22	0.88	0.09	2.34
	15	37	0.70	296.0	117.7	178.3	19.0	36.1	0.21	0.82	0.09	2.22
	20	43	0.70	396.0	166.8	229.2	19.5	37.7	0.21	0.76	0.09	2.30
	25	47	0.70	496.0	215.8	280.2	19.2	38.7	0.21	0.70	0.09	2.44
	30	51	0.70	596.0	264.9	331.1	19.2	39.7	0.21	0.64	0.09	2.62

Table 5. Results of determining attenuation coefficients γ_{eq1} and γ_{eq2} for sand at the study site.

Layer	z (m)	R_u	I_{red}	φ (^{0})	$φ_{red}$ (^{0})	N_q	$N_{q(eq)}$	γ_{eq1}	γ_{eq2}
Layer 1	4	0.10	0.90	29.7	27.2	17.8	13.4	0.76	0.90
	6	0.10	0.90	31.6	29.0	22.1	16.4	0.74	0.90
Layer 2	8	0.06	0.94	34.3	32.7	30.5	25.0	0.82	0.94
	10	0.06	0.94	34.9	33.3	32.8	26.9	0.82	0.94
	15	0.06	0.94	36.1	34.4	38.1	30.9	0.81	0.94
	20	0.06	0.94	37.7	36.0	47.1	37.8	0.80	0.94
	25	0.06	0.94	38.7	37.0	54.0	43.0	0.80	0.94
	30	0.06	0.94	39.7	38.0	61.8	48.9	0.79	0.94

Table 5 shows that the attenuation coefficient of tip resistance γ_{eq1} in saturated sand decreases with depth because the friction angle of the soil increases with depth; the attenuation coefficient of friction γ_{eq2} depends only on I_{red} or R_u. Fine-silty sand in Layer 1 has γ_{eq1} = 0.74÷0.76 and γ_{eq2} = 0.90. Fine sand in layer 2 has γ_{eq1} = 0.79÷0.82 and γ_{eq2} = 0.94. These coefficients are suitable for driving piles, disregard of soil disturbance when constructing bored piles. The obtained results are quite agreement with the ones determined by empirical method according to the standard TCVN 10304:2014. Some of the values are different because the soils is classified into only two states of "medium dense sand" and "dense sand" in the empirical method, which is a fairly wide range when the SPT value can vary from 10 to 30 blows with medium dense sand and from 30 to 50 blows with dense sand. On the other hand, the determination of FSL from the chart also has a certain deviation.

4. Conclusions and recommendations

The design of earthquake-resistant pile foundation should be applied to buildings of grade III or higher, which are located in the area with a_g ≥ 0,08 g, especially in a saturated sandy ground. There are many methods of determining the attenuation coefficients for seismic design. With saturated sandy soil, besides the empirical method (TCVN 10304:2014), it is also possible to use SPT for estimation of attenuation coefficients based on values of I_{red} or R_u. The results of applying this method in Nhon Hoi economic zone show that: medium dense, fine-silty sand has a attenuation coefficient of tip resistance γ_{eq1} = 0.74÷0.76; attenuation coefficient of friction γ_{eq2} = 0.90; dense fine sand has γ_{eq1} = 0.79÷0.82; γ_{eq2} = 0.94. The two methods applied in this paper are indirect methods. To improve the reliability, it is necessary to combine with simulated impact pile test or a triaxial test with simulated earthquake load to determine directly the increase of R_u for each type of soil.

Author contributions

Phong Van Nguyen proposes ideas and contributes to the manuscript; Ngoc Ba Thai constructs the manuscript and contributes to the material analyses. The authors declare no conflict of interest.

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