PROCEEDINGS OF THE 21st SOUTHEAST ASIAN GEOTECHNICAL
CONFERENCE AND 4th AGSSEA CONFERENCE (SEAGC-AGSSEA 2023)
BANGKOK, THAILAND, 25-27 OCTOBER 2023

In an era of increasing urgency to protect geo-structures from natural disasters and environmental threats, importance of geotechnical profession has never been more pronounced. Ensuring safety, cost-effectiveness, and environmental sustainability are paramount. This demand for innovation in geotechnology resonates regionally and globally as we tackle these challenges together.

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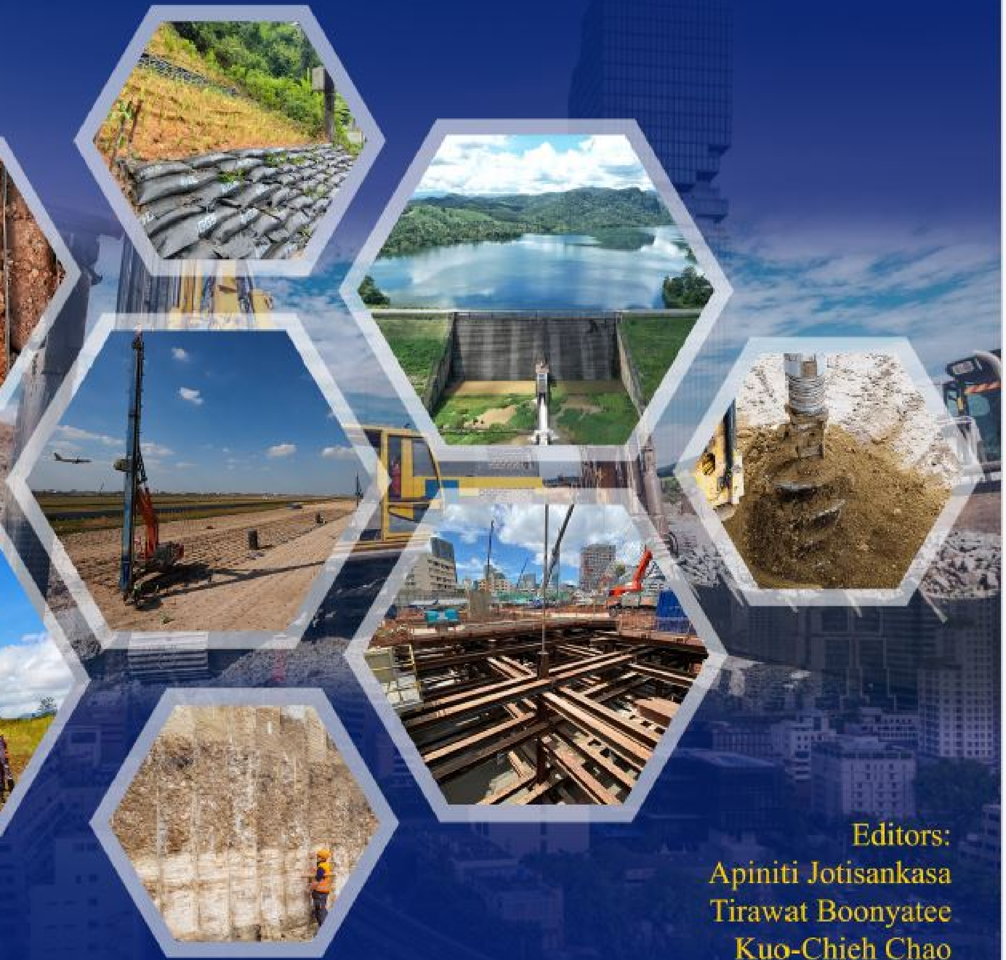
Innovative geotechnology to meet new challenges in the region and beyond



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Organized by

South East Asian Geotechnical Society (SEAGS)

Engineering Institute of Thailand under H.M. The King's Patronage

Thai Geotechnical Society (TGS)

Under the auspices of:

International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE)

Session 6A: Ground Improvement 3

26 October 2023 Session 6A: Ground improvement 3			
Time:	13:45 - 15:00	Room:	World Ballroom A
Chairperson(s):	Prof. Dr.Pitthaya Jamsawang		Dr.Chana Phutthananon
PVD's to dissipate excess pore pressures during Dynamic Compaction of Clayey Sands with 30-45% Fines Content			
Paper ID:6679	Francisco Baez, Phatthana Pansaen and Jansen Rementilla		
Study the Behavior of Excess Pore Water Pressure in Sand Subjected to Dynamic Compaction			
Paper ID:8314	Chayakorn Jangsakul, Susit Chaiprakaikeow, Suriyon Prempramote and Suttisak Soralump		
Numerical Back Analysis from Observational Method for Consolidation Settlement due to the Preloading			
Paper ID:9957	Mahabub Sadiq		
Investigation on Strength Development Characteristics of Improved Soil Based on the Relationship Between Consistency and Shear Strength			
Paper ID:6105	Yusei Matsuo, Hiruhumi Usui, Takenori Hino and Toshiyuki Himeno		

Study the Behaviour of Excess Pore Water Pressure in Sandy Soil Mixed with Fine Content by using High Vacuum Densification Method (HVDM)

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ABSTRACT: High Vacuum Densification Method (HVDM) is a soil improvement technique and is a combination of Dynamic Compaction Method (DC) and Vacuum Consolidation Method (VCM). The purpose of this technique is to increase the bearing capacity and reduce the settlement by using a tamper that freely drops at a designed height with vacuum system. However, during the soil improvement process with DC, excess pore water pressure may increase, and it may take a long period of time for the dissipation of excess pore water pressure. This research aims to analyse a suitable model for predicting the changes in excess pore water pressure that occur during the soil improvement process with DC in order to understand its behaviour before beginning of actual work. The study area is a runway and taxiway at U-Tapao International Airport, 2nd phase, Rayong Province, Thailand, where piezometers and accelerometer were installed to measure pore water pressure and ground acceleration during the tamping. Most of the soil in study area is sandy soil mixed with some fine content. A Mohr-Coulomb model was used in PLAXIS 2D software for the analysis. The field result showed that excess pore water pressure decreased with increasing depth and horizontal distance while the peak ground acceleration decreases with the increase of horizontal distance from drop point. While comparing the excess pore water pressure results from field and modelling, the maximum different value is 2.43 kPa.

KEYWORDS: High Vacuum Densification Method, Dynamic Compaction, Pore Water Pressure, Numerical Analysis

1. INTRODUCTION

There are several engineering soil improvement techniques that can be used to enhance the engineering quality of soft soil such as (a) prefabricated vertical drains (PVDs) and fill preloading, (b) vacuum consolidation together with PVDs, (c) stone columns, (d) thermal treatment, (e) chemical mixing, (f) electro-osmosis, and (g) deep dynamic compaction (Tabatabaei 2014). Although there are various methods for soil improvement, prefabricated vertical drains (PVD) with preloading is used in most of the construction works in Thailand but this method takes long time to complete (Khattiwong, 2015). Likewise, High Vacuum Densification Method (HVDM) was invented in China and is the combination of dynamic compaction method and vacuum consolidation method. This method takes shorter time to complete as compared to prefabricated vertical drains (PVD) with preloading method (Liang and Xu 2010). Although dynamic compaction is effective for the mitigation of liquefaction (Thevanayagam et al. 2009), the excess pore water pressure will be created during the soil improvement process and it might lead to liquefaction of sand or silty soils (e.g., Roesset et al. 1994; Majdi et al. 2007; Cui 2010). Consequently, that liquefaction will make dynamic compaction less effective in terms of densification of the subsoil (Nashed et al. 2009a, b). Therefore, this research focuses on analysing suitable models to understand the response to excess pore water pressure generated during soil improvement process.

2. METHODOLOGY

2.1 Study Area Conditions

The study area, U-Tapao International Airport, is located approximately 190 kilometres away in Southeast part of Bangkok in Rayong Province. In addition, U-Tapao International Airport is also located near the deep-sea port of Map Ta Phut and Chuk Samet port (Figure 1). The project intends to construct runways and taxiways in the area. Most of the soil in this study area is sandy soil mixed with fine content and has a low bearing capacity. Therefore, it is necessary to improve their strength in order to construct the runways and taxiways in this area.

2.2. High Vacuum Densification Method (HVDM)

HVDM is a method of soil improvement technique that combines the knowledge of two techniques, namely Dynamic compaction and Vacuum consolidation. This method is suitable for large-scale projects as it avoids the need for extensive soil excavation, which can be costly. Dynamic compaction is performed to increase the soil density and create positive pressure while Vacuum consolidation aims to create negative pressure. By creating a pressure difference, excess water pressure in the soil can be dissipated rapidly and various principles can be demonstrated as shown in Figure 2.

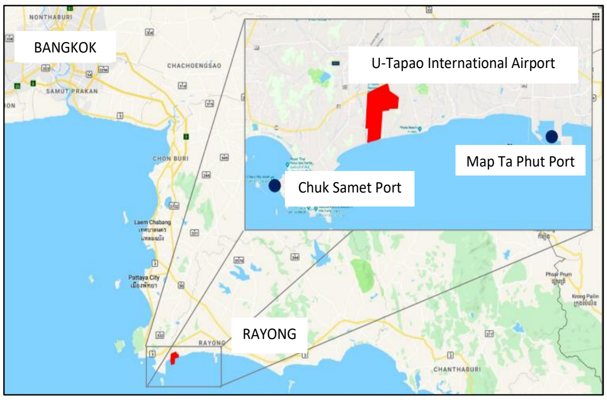

Figure 1 Location of U-Tapao international airport

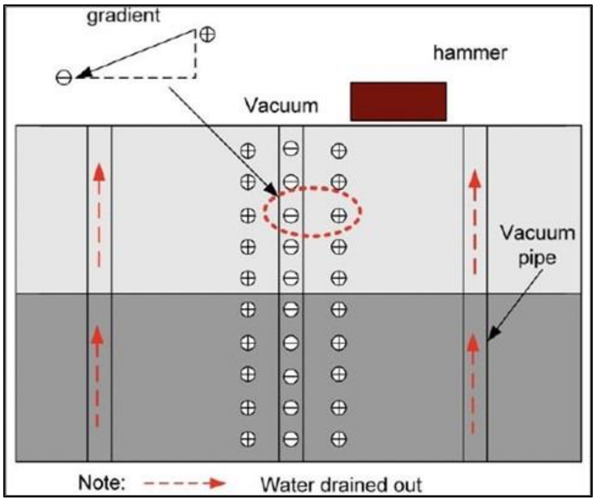

Figure 2 The mechanism of HVDM (Ji-hong 2014)

Table 1 Soil parameters for the modelling

Depth		Soil Type	q_c (MPa)	F_s (kPa)	γ (kN/m^3)	Void Ratio	S_u (kPa)	E(kPa)	Poisson ratio	G/G_{max}	Damping Ratio
From	To										
0.0	0.8	Silty Sands 1 (Very Dense)	3.0	33.3	17.70	1.139	59.0	1.83E+05	0.35	0.0081	0.26
0.8	2.0	Silty Sands 2 (Loose)	1.3	19.4	16.50	1.532	33.3	1.26E+05	0.35	0.0079	0.26
2.0	2.3	Clay 1 (Medium Stiff Clay)	0.7	48.4	17.60	1.163	46.3	2.33E+05	0.50	0.0112	0.24
2.3	3.2	Silty Sands 3 (Very Loose)	1.1	22.0	16.50	1.522	29.9	1.97E+05	0.35	0.0138	0.24
3.2	3.6	Clay 2 (Stiff)	1.3	100.7	18.70	0.897	79.5	3.27E+05	0.50	0.0110	0.24
3.6	6.7	Silty Sands 4 (Medium Dense)	10.1	246.8	20.10	0.633	195.2	5.39E+05	0.35	0.0140	0.23
6.7	10.0	Sand (Dense)	14.2	247.2	20.40	0.584	231.5	5.47E+05	0.35	0.0124	0.24

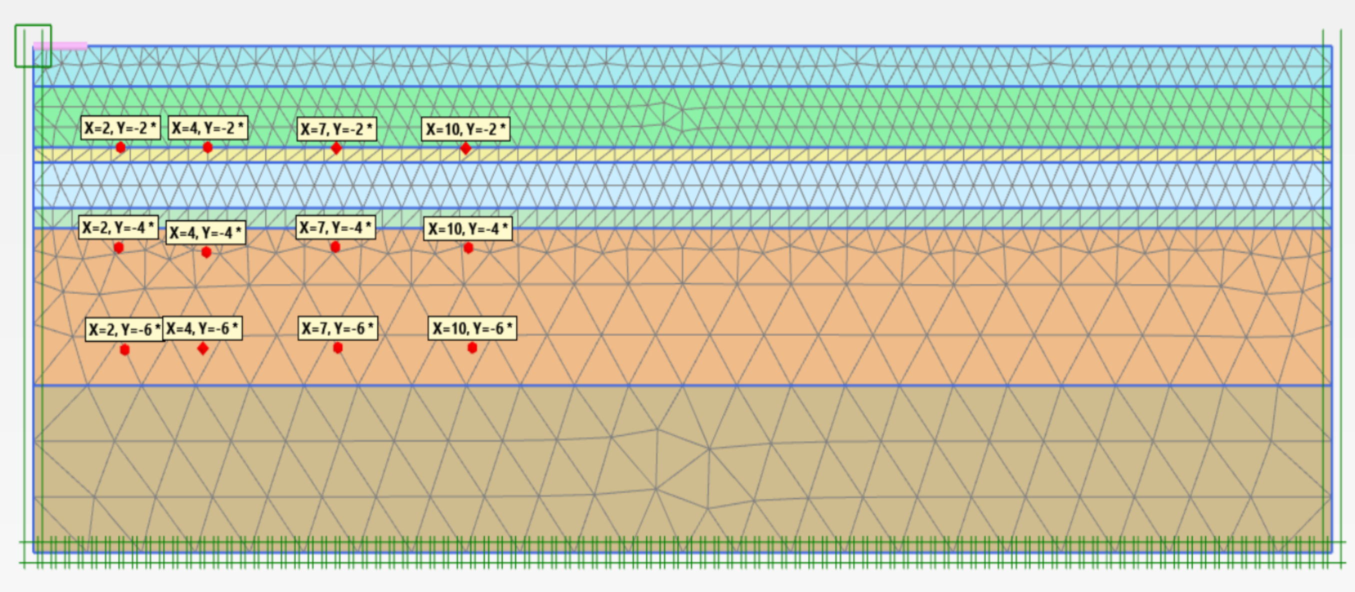

Figure 3 Model and mesh for simulation by PLAXIS 2D

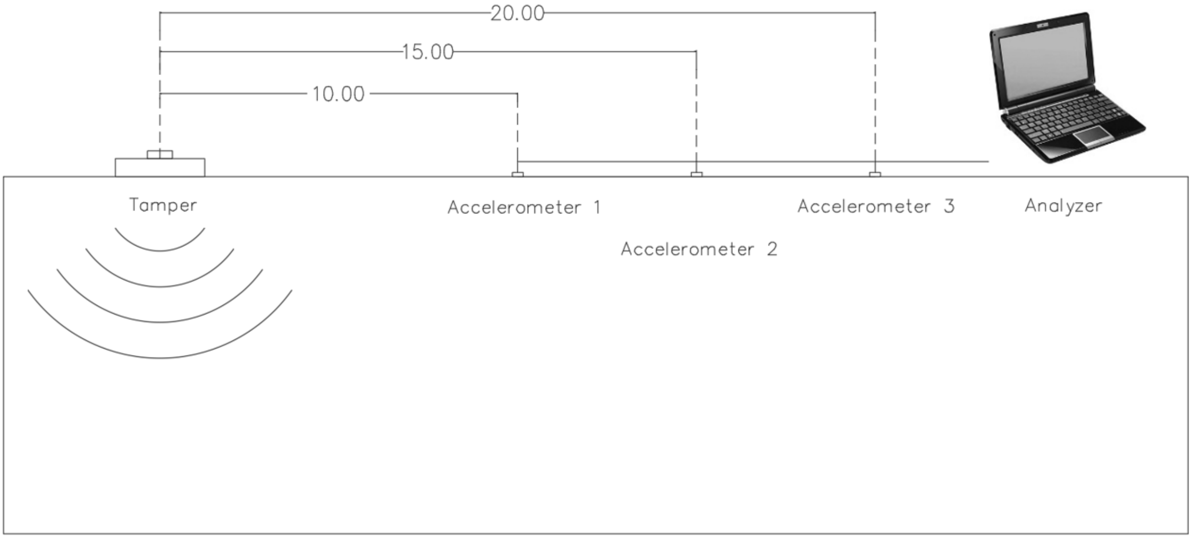

Figure 4 Ground acceleration testing

2.3 Field Testing and Measurement

Two types of soil investigation works were carried out in the project area i.e., Cone penetration Test (CPT) and Spectral Analysis of Surface Waves (SASW). Likewise, 2 field measurements were carried out in the study area including pore water pressure generation measurement and ground acceleration testing due to tamping. For pore water pressure generation measurement, 3 piezometers were installed in the study area at 2-, 4- and 6-meters depth in order to measure pore water pressure generated during the improvement process. Ground acceleration testing was performed by using accelerometers at 10, 15 and 20 meters from the center of drop point as shown in Figure 4.

3. THE RESULT AND DISCUSSION

3.1 Soil Investigation

The result of Cone Penetration test and SASW is shown in Figure 5 and Figure 6 respectively.

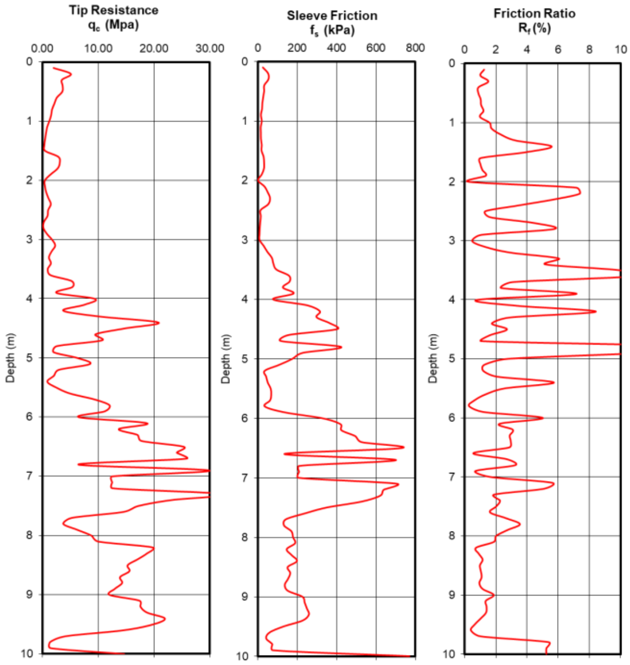

Figure 5 Result of Cone Penetration Test (CPT)

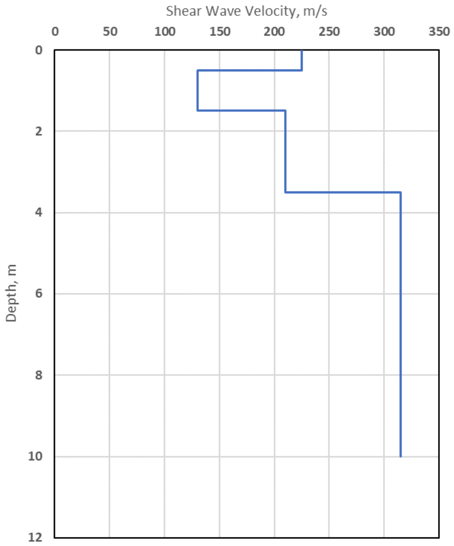

Figure 6 Result of Spectral Analysis of Surface Waves (SASW)

3.2 Pore Water Pressure Generation

Three Piezometers were used for measurement of pore water pressure during soil improvement process. They were installed at 2, 4, and 6 meters depth. During improvement process, the maximum excess pore water pressure generation with increasing of horizontal distance from the centreline of drop point is illustrated in Figure 7.

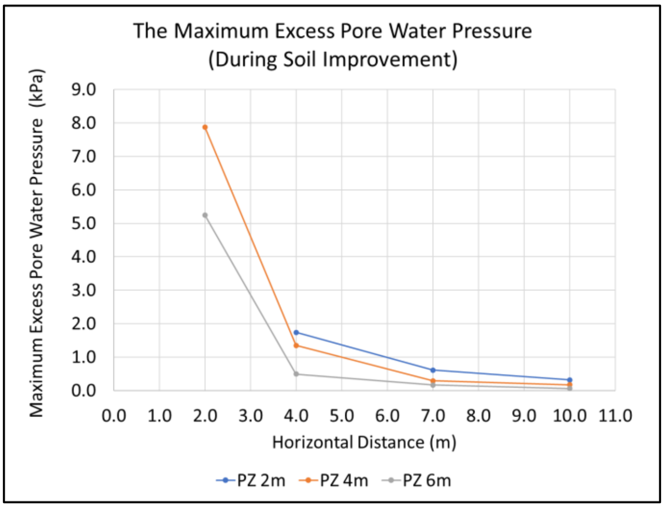

Figure 7 Plot of excess pore water pressure with horizontal distance from drop point

3.3 Ground Acceleration Testing

Ground acceleration testing was conducted during tamping by using accelerometers. Accelerometers were put on ground surface at 10, 15 20 meters from centreline of drop point. The result of ground acceleration testing is illustrated in Figure 8. The summary result of peak ground acceleration with horizontal distance from drop point is presented in Table 2.

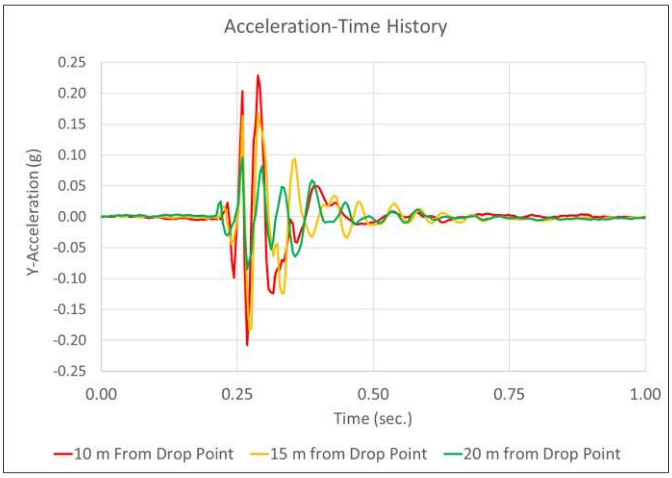

Figure 8 Time history plot of Y-acceleration

Table 2 Peak Y-Acceleration

Horizontal Distance from Drop Point (m)	Peak Acceleration. (g)
10.00	0.2290
15.00	0.1830
20.00	0.0958

3.4 Simulation Modelling

3.4.1 Interpretation of soil investigation

Two-dimensional finite element analysis was performed using PLAXIS 2D software. Axisymmetric method was used for analysis and the dimension of the model taken was 30 m x 10 m with ground water level present at 1.50m from the ground surface. The Mohr-Coulomb Method was used for the analysis and the soil parameters for the modelling were obtained from soil investigation in the study area by Cone Penetration Test (CPT). The soil classification has been carried out using Figure 9 and conversion of CPT values to other parameters is carried out using Equation 1-7 and is shown in Table 1. The model and mesh for simulation using PLAXIS 2D is shown in Figure 3. To avoid the wave reflection on the model boundary, viscous boundary was specified at X_{max} and Y_{min}.

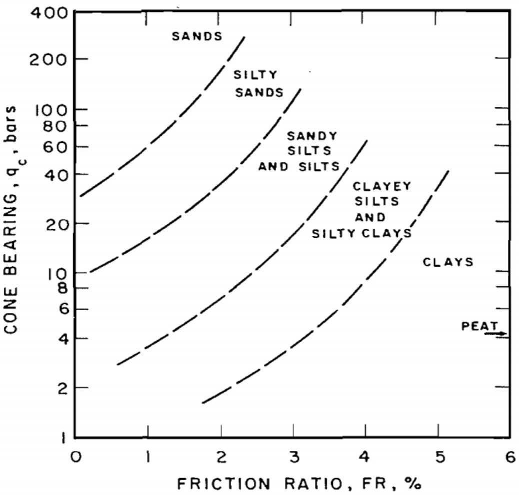

Figure 9 Simplified soil classification chart for standard electric friction cone

For unit weight based on Robertson (2010)

$$\gamma/\gamma_w = 0.27\log R_f + 0.36\log(q_t/P_a) + 1.236 \quad (1)$$

For undrained Shear Strength, Su, based on Terzaghi & Peck (1967)

$$S_u = 6.25N \quad (2)$$

For aproximated q_c/N based on Robertson &Campanella (1983)

Soil Type	Approximate q_c/N
Sand	5.0
Silty Sand	3.3
Sandy Silt and Silt	2.4
Clayey Silt and Silty Sand	1.8
Clay	1.0

For Void ratio from phase diagram

$$e = \frac{\gamma_t - \gamma_w G_s}{\gamma_w S - \gamma_t} \text{ When } G_s = 2.65 \text{ for Sand and } 2.70 \text{ for Clay} \quad (3)$$

For Young's Modulus

$$E = 2\rho V_s^2\,(1+\nu) \quad (4)$$

For Poisson ratio based on Davidovici et. al. (1985)

ν = 0.35 for Silty and Clayey Sand

= 0.50 for Saturated Clay

For Shear Modulus

$$G = \frac{E}{2(1+\nu)} \quad (5)$$

Maximum Shear Modulus Based on Anbazhagan et. al. (2010)

$$G_{max} = 24.28N^{0.55} \quad (6)$$

Damping Ratio Based on Ishibashi and Zang (1993)

$$D = 0.167\left(1 + e^{-0.0145 I_p^{1.3}}\right)\left\{0.586\left(\frac{G}{G_{max}}\right)^2 - 1.547\left(\frac{G}{G_{max}}\right) + 1\right\} (7)$$

3.4.2 Impact Stress during Dynamic Compaction

The dynamic stress-time function for dynamic compaction can be assumed as triangular impulse loading (Mayne and Jones, 1983) as shown in Figure 10. The peak dynamic stress (σmax) and time duration (Δt) has been obtained using equation 8 and 9 respectively and the result of dynamic stress calculation is as shown in Table 3.

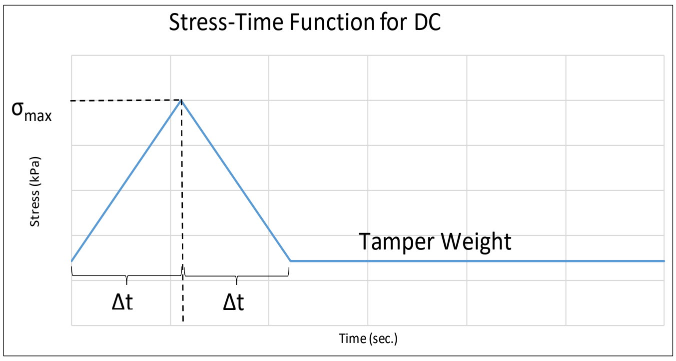

Figure 10 Dynamic stress-time function

$$\sigma_{max} = \sqrt{\frac{32WHGr_o}{\pi^2(1-\nu)}} \cdot \frac{1}{\pi r_o^2} \quad (8)$$

$$\Delta t = \frac{2W\sqrt{2gH}}{(\pi r_o^2) g \sigma_{max}} \quad (9)$$

Table 3 Parameters for the applied load

Symbol	Description	Value
H	Drop Height (in m)	10.0
g	Acceleration of Gravity (in m/s2)	9.81
W	Weight of Tamper (in kN)	140
r_o	Radius of Tamper (in m)	1.25
G	Shear Modulus of Soil (in MPa)	145.8
ν	Poisson Ratio	0.35
σ_{max}	Peak Dynamic Stress (in kPa)	7,268
Δt	Load Time (in sec.)	0.01
Self-Weight	Weight of Tamper (in kPa)	28.50

3.5 Computation Result

To validate the applied load and modelling, the comparison between field testing result and computation results was performed. Figure 11, Figure 12, Figure 13 and Figure 14 illustrates the plot of excess pore water pressure generation between field and Modelling. For the comparison, the different value can be calculated by using Eq.12 and the result of calculations shown in Table 4.

$$\text{different value} = Modeling - Field \tag{12}$$

Table 4 Different value of excess pore water pressure generation between field and Modelling

Depth (m)	Different value (kPa)			
	X = 2 m	X = 4 m	X = 7 m	X = 10 m
-2.0	N/A	1.16	0.09	0.01
-4.0	-0.78	2.10	1.06	0.54
-6.0	-1.15	2.43	1.35	0.77

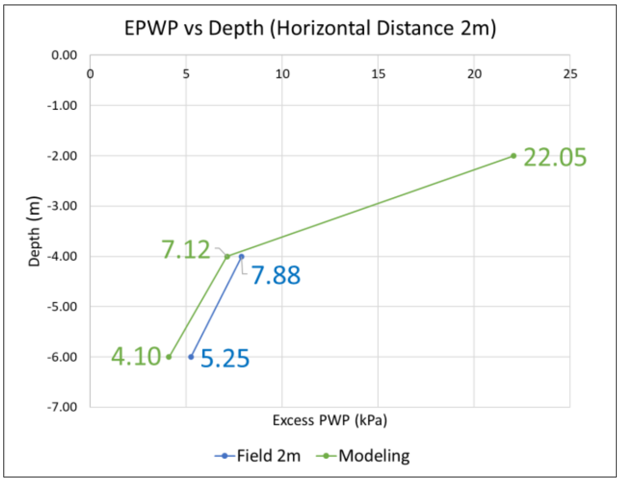

Figure 11 Excess pore water pressure from field testing and modelling (horizontal distance 2 meters)

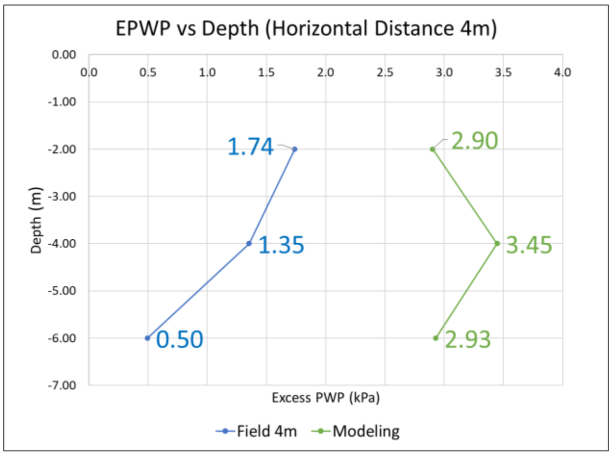

Figure 12 Excess pore water pressure from field testing and modelling (horizontal distance 4 meters)

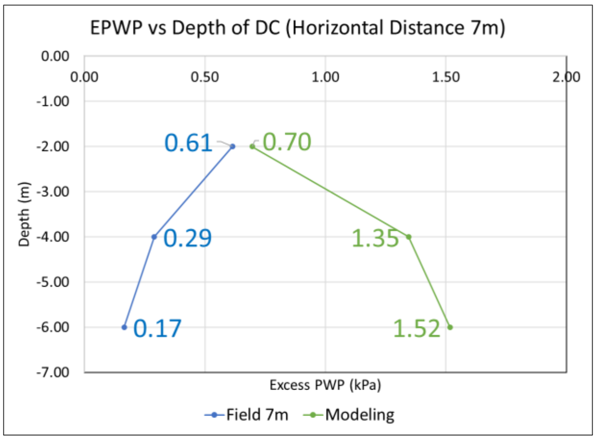

Figure 13 Excess pore water pressure from field testing and modelling (horizontal distance 7 meters)

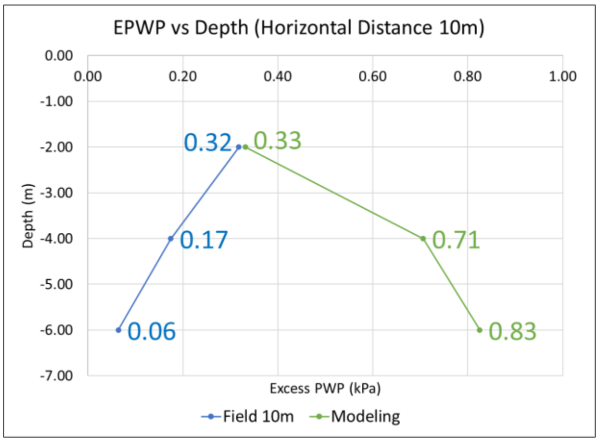

Figure 14 Excess pore water pressure from field testing and modelling (horizontal distance 10 meters)

4. CONCLUSIONS

In analysing the excess pore water pressure from the field relative to the horizontal distance from the drop point, it becomes evident that there exists a discernible trend of decreasing excess pore water pressure with increasing horizontal distance. Furthermore, it is also observed that the peak acceleration exhibits a gradual decrease with the increase in horizontal distance. This is in accordance to the law that impact energy decreases with distance.

Based on the modelling outcomes, it was observed that the presence of excess pore water pressure decreased with increase in depth at a horizontal distance of 2 meters. However, it was found that the excess pore water increased with depth when the horizontal distance was shifted to 4, 7 and 10 meters.

Additionally, when comparing the field results with the modelling outcomes, it was observed that the maximum different value of excess pore water pressure is 2.43 kPa.

5. ACKNOWLEDGMENTS

We extend our sincere gratitude to the Department of Civil Engineering, Faculty of Engineering, Kasetsart University, as well as Geotechnical Engineering Research and Development Centre (GERD) and Geoharbour Construction Co., Ltd., for their supporting and facilitation throughout the course of this study. Furthermore, we would like to express our appreciation to all individuals who contributed to this research, albeit not explicitly mentioned in this article.

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