Soil Improvement by Dynamic Compaction of Various Trial Techniques on Sandy Soil with High Fines Content and High Ground Water Table

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ABSTRACT: Two types of dynamic compaction, Rapid Impact Compaction (RIC) and High Vacuum Densification Method (HVDM), were selected for soil improvement during the construction of the 2nd runway and taxiway of U-Tapao International Airport. The improvement was targeted to increase soil strength by higher than 8% of California Bearing Ratio (CBR) from topsoil down to 6-meters depth. A preliminary field test was conducted to find out that the site consisted of loose sand with 20-40% of fines content and a high-water table. The main objective of the research was to study the effectiveness of RIC and HVDM in such unfavourable conditions. The main difficulty of improving soil in such condition is that the excess pore water pressure would dissipate at a slow pace, especially at soil layer with high fines content, that could reduce the effectiveness of the compaction and the soil improvement would take long period of time. Piezometer was installed at 2 meters, 4 meters, and 6 meters depth and were used to observe the generation and dissipation of pore water pressure during compaction. CPT and SASW tests were carried out before and after the dynamic compaction to observe the changes in soil strength after compaction. The study showed that liquefaction potential is present on-site during compaction at 2 meters depth. The waiting period of compaction to move to the next pass for RIC is approximately 3 days and for HVDM method is approximately 5-7 days. The improvement depth for RIC method is up to 2.5 meters with the maximum improvement depth of 5 meters and for HVDM method is up to 3.5 meters depth with the maximum improvement depth of 5.5 meters.

KEYWORDS: Dynamic compaction, Rapid impact compaction, High vacuum densification method, and Soil Improvement.

1. INTRODUCTION

Dynamic Compaction (DC) was first introduced by Louis Ménard in 1970s (Menard and Broise, 1975). Initially the preferred soil condition for this method was soils with natural sandy gravel properties (Chow et al, 1992; Nashed et al, 2009), but then people also used the method for alluvial, saturated clay, and silty ground (Chen et al, 1998; Nashed et al, 2009; Yao et al, 2022).

The impact from the compaction generates excess pore water pressure. The excess pore water pressure may absorb or supress the compaction energy into travelling to the lower depth of the soil body which decrease the effectiveness of the improvement. To make the most of soil improvement, compaction needs to wait until the excess pore water pressure dissipates from the soil. However, the more fines content in the soil, the longer one must wait for it to dissipate.

To overcome this matter, new techniques for the dynamic compaction has been developed i.e Rapid Impact Compaction (RIC) and High Vacuum Densification Method (HVDM). RIC is a middle-deep improvement of the ground that fills the gap between surface compaction method and deep compaction method (Tarawneh and Matraji, 2014; Avalle et al, 2019). Recently, the RIC method was used in a research study for a typical hydraulic fill reclaimed line with seabed sediments as the main filling (Cheng et al, 2021). HVDM technique was used to improve soil that has high fines content, such as sandy silt and sandy clay soil (Liang and Xu, 2010; Ji-Hong, 2014). The research study conducted in Hangzhou Bay (Zhong and Haryono, 2019) used HVDM method to improve reclaimed fill consisting of silts (16.5% clayey content) with thickness of 2.2-7 m and saturated silt with thickness of 15 to 17 m. Before and after improvement CPT result showed an increase up to 8 m depth.

The objective of this research is to study the effectiveness of RIC and HVDM by observing the excess pore water pressure and the increase in soil strength after improvement. The test site is in Rayong Province, Thailand. The research area was a part of the project for the U-Tapao International Airport runway and taxiway construction.

2. METHODOLOGY

2.1 Soil Conditions

The research area is named MRO 2 which will be constructed as an airplane runway. The plan of the project was to improve the soil up to 6 meters depth having California Bearing Ratio (CBR) greater than 8% or equal to 120 m/s for shear wave velocity (Rosyidi et al, 2006).

Two trial area (Trial Area 1 and Trial Area 2) for RIC method and two trial area (Trial area 71 and Trial Area 78) for HVDM method were prepared for the study. The soil properties of the trial area are shown in Table 1. Sieve result of the RIC and HVDM trial area shows that 20-40% of fines content is present in the soil (Figure 1). The general soil profile of each trial area is shown in Figure 2. The ground water table was found 1.5 meters below the ground surface.

Table 1 Soil properties of RIC and HVDM trial area

Location	Wn (%)	LL	PL	PI	Fines Content (%)	Permeability (cm/sec)
Trial Area 1	10	30.0	15.6	14.4	30	-
Trial Area 2	12.2	44.9	21.9	23.0	22	-
Trial Area 71	11.8	31.7	19.9	11.8	37	$6.41 * 10^{-6}$
Trial Area 78	12.3	31.2	22.3	8.9	38	$3.04 * 10^{-6}$

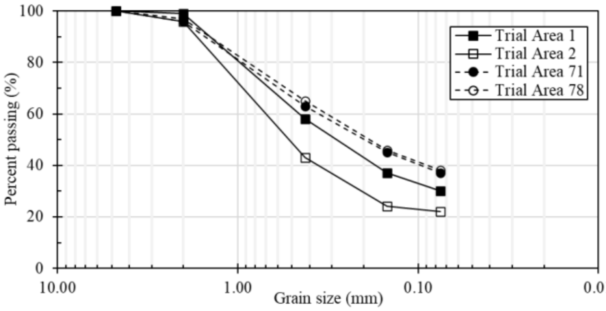


Figure 1 Sieve analysis result of RIC and HVDM trial area

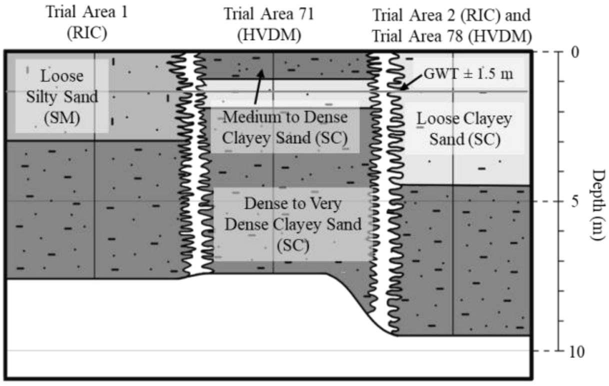


Figure 2 General soil profile of RIC and HVDM trial area

2.2 Equipment and testing program

There are 2 different testing programs, the RIC method and the HVDM method. RIC method uses 2 passes, 6 m spacing for 1st pass and 3 m spacing for 2nd pass. The RIC machine (as shown in Figure 3) used 9-ton hammer having the diameter of 1.5 m dropped from 1 meter height in to the 2-ton steel foot that is always in contact with the soil surface. Maximum blow per point was set at 30 blows and during compaction the blow will be stopped if the compaction settlement reached 80 cm.

Figure 3 RIC machine

The HVDM method uses 2 passes as well, 7x8 spacing for both 1st and 2nd pass. In this method, the 14-ton hammer having the diameter of 2.5 m was dropped from 10 meters height into the ground surface. The vertical steel galvanized pipes were installed with 4x3.5 meters spacing for the 3 meters depth and 8x3.5 meters spacing for the 6 meters depth. These vertical pipes were connected to horizontal pipes which was then connected to a vacuum pump as shown in Figure 4. The maximum blow of the compaction is 2-3 blows.

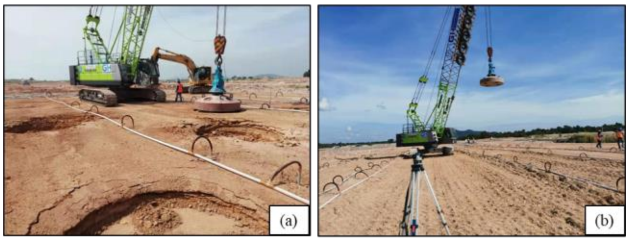


Figure 4 Typical HVDM set-up at construction site

2.3 Field test and monitoring instrumentation

The field tests consisted of CPT and SASW test. Each of the test were conducted before and after compaction, therefore the increase of the soil strength can be observed by comparing the test results. These field tests were aimed for 10 meters depth.

The CPT was carried out before and after the improvement. Initial and final soil strength can be detected and each method's effectiveness in improving the soil can be observed. Another information that can be obtained from CPT test result is soil classification by Robertson and Campanella (Robertson and Campanella, 1983) for standard electric friction cone (Figure 5). In this soil classification, the soil can be interpreted as sands to clays from the tip resistance (q_c) and friction ratio data from CPT results. Therefore, the information about several soil types that are present below the ground at CPT location and the thickness of that soil type can be analyzed from the CPT data.

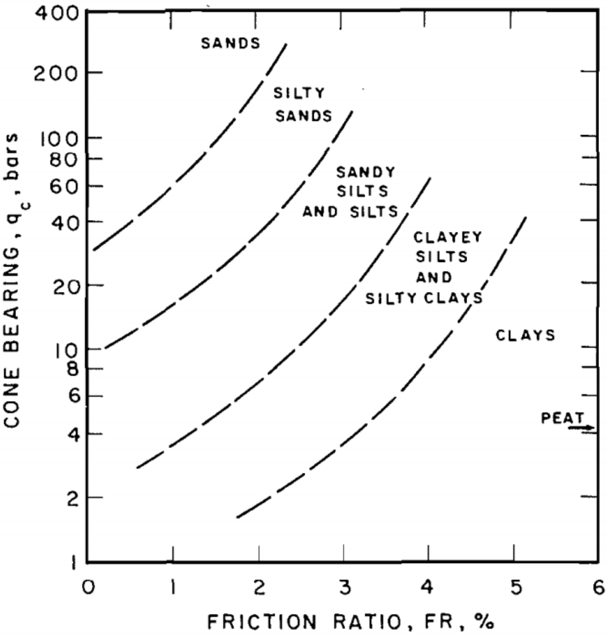


Figure 5 Soil classification chart for standard electric friction cone (adapted from Robertson and Campanella, 1983)

SASW test has been applied by researchers of the University of Texas at Austin to investigate the Vs and shear modulus of pavement and highway materials (Nazarian and Stokoe, 1985; Nazarian et al, 1988; Ismail et al, 2012). Many SASW test were implemented in Thailand for measuring the Vs of a dam, dyke, and liquefaction site (Mase et al, 2018; Soralump et al, 2021; Shrestha et al, 2019) and studying the small strain modulus of the silty sand subgrades (Barus et al, 2019).

In this research study, the location of the SASW tests were done near the CPT location at each trial area as shown in Figure 4 to compare the two test methods results. The SASW test used receiver with spacing of 1, 2, 4, 6, and 10 m and used common receiver mid-point configuration for the test, which are shown in Figure 6 below.

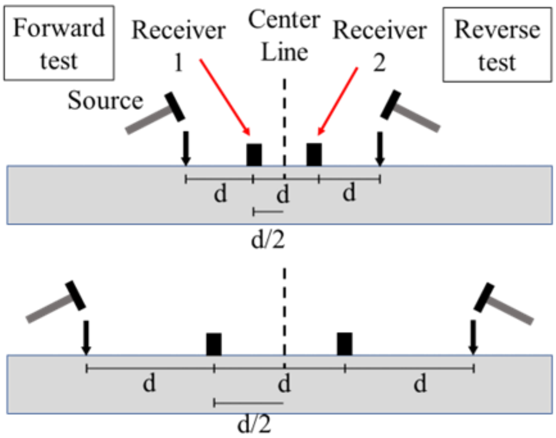
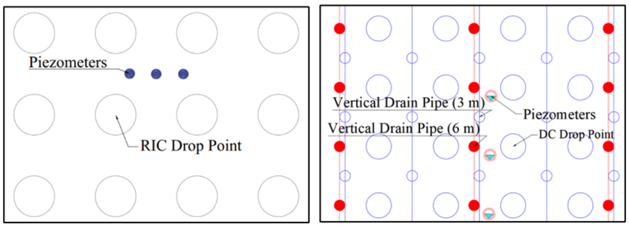


Figure 6 The common receiver mid-point test configuration that performed in this study (adapted from Alditra et al, 2020)

Impact source of 100 kg drop weight hammer was used in this study area at far receiver spacing of 6 m and 10 m to generate a low frequency wave to investigate deep profile, and a sledgehammer was used for the intermediate and short spacing. Two 2-Hz geophones were used as the receivers. The SASW test consisted of the following procedures: firstly, the data that were recorded by the spectrum analyzer will be obtained from the field, and then to evaluate the dispersion curves and shear-wave velocity profile using the WinSASW program developed by Joh in 1996 (Joh and Stokoe, 1997)

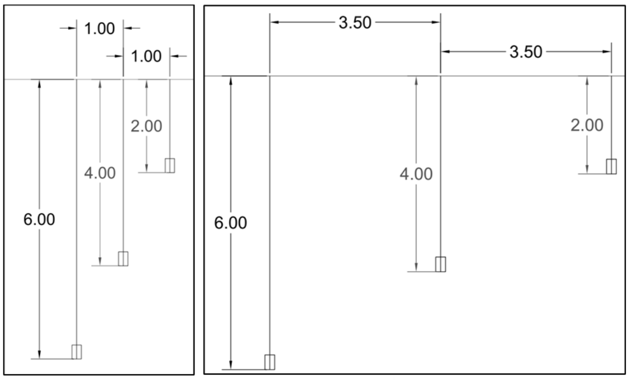
The dynamic of pore water pressure is an essential parameter in the dynamic compaction method. Excess pore water pressure can reduce the effectiveness of compaction as it disperses impact energy to a deeper layer. The dissipation time of excess pore water pressure could determine the next pass of the compaction. Therefore, in this research study vibrating wire piezometers were placed in each trial area. Each trial area had a piezometer placed at 2-meters, 4-meters, and 6-meters depth below the ground surface as shown in Figure 7 and Figure 8. The piezometers were carefully installed between the drop points to reduce the risk of the piezometer being damaged by the impact. The data reading frequency was set each time after the machine finished compacting one point for the RIC method and each time the hammer was dropped for the HVDM method. In addition, supplementary piezometer reading was set at increasingly longer interval (a few minutes at first then a few hours adjusting with the reducing speed of excess pore water pressure dissipation) and once a day when no compaction work was being conducted in the trial area to obtain pore water pressure data after compaction was done for some amount of time.

Extreme care should be taken with piezometer equipment when compaction occurs. Piezometer pieces of equipment that are located on the ground surface, for example, a data logger and a connecting cable between the piezometer sensor and the data logger, are very vulnerable to heavy moving equipment when compacting. Some ways to protect the equipment are to erect barriers so that the compacting machine does not pass by in the area near the piezometer sensor and build a platform slightly higher than the ground level so that the equipment is more easily seen and does not stagnate on the ground when heavy rains occur.



(a) (b)

Figure 7 Piezometer location of RIC (a) and HVDM (b) trial area



(a) (b)

Figure 8 Installation depth and spacings of piezometers of RIC (a) and HVDM (b) trial area

3. RESULTS AND DISCUSSION

The installed piezometers were able to record pore water pressure generation and dissipation over time. The data was then used to calculate and analyze excess pore water pressure dissipation. The excess pore water pressure ratio was analyzed to comprehend the liquefaction potential during compaction and to know the waiting period for both methods. The maximum depth of improvement for each method is observed by analyzing the existing field test.

3.1 Excess pore water pressure

The piezometers installed in the trial area were able to record the change of pore water pressure over time from trial area 2 (RIC method) and trial area 78 (HVDM method). The pore water increases at 0 hours was the 1st pass of each method. The pore water increases at 48 and 72 hours was the 2nd pass of the RIC and the HVDM method, respectively. Excess pore water pressure dissipation curves were generated by subtracting the highest value pore water pressure with in-situ pore water pressure. The curves for excess pore water pressure for the RIC and HVDM method are shown in Figure 9 and Figure 10 respectively. The RIC method produced higher excess pore water pressure during the 2nd pass because the spacing between drop points was 3 meters wide, which is shorter compared with the 1st pass, which was about 6 meters wide. The HVDM method produced almost the same excess pore water pressure during the 1st and 2nd passes because each pass used the same spacing for the drop point, which was 7x8 meters.

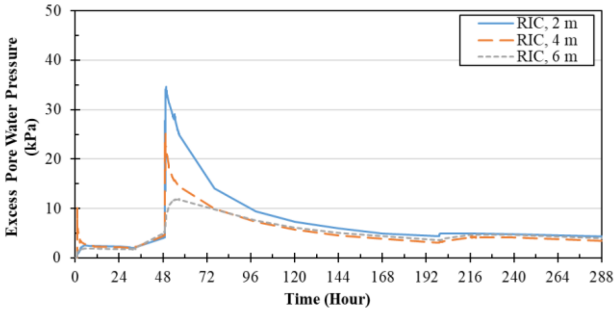


Figure 9 Excess pore water pressure (kPa) dissipation over time at 2-meter, 4-meter, and 6-meter depth at Trial Area 2 (RIC method)

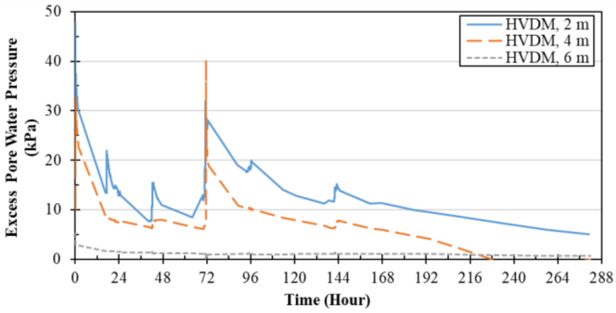


Figure 10 Excess pore water pressure (kPa) dissipation over time at 2-meter, 4-meter, and 6-meter depth at Trial Area 78 (HVDM method)

The maximum excess pore water pressure produced at 2-meters depth from the RIC method was about 33 kPa and the HVDM method was about 47 kPa, where the latter shows higher value. The maximum excess pore water pressure produced at 4-meters depth from the RIC method was about 25 kPa and the HVDM method was about 40 kPa, where the latter shows higher value. The HVDM method were able to produce higher excess pore water pressure because it generates higher energies at one single impact point due to the difference in the hammer weight, hammer diameter, and drop height. Each blow from the RIC method was only able to produce 10.8 T.m, which is less compared to each blow from the HVDM method that produced 140

T.m for each blow. Both RIC and HVDM are shown to produce a higher value of the ratio over time at a 2-meter depth compared with the deeper layers (4-meter and 6-meter depth). It can be concluded that the upper layer of soil had the biggest impact from the compaction and the energy influence seems to decrease over deeper layers of soil.

3.2 Excess pore water pressure ratio

The excess pore water pressure ratio (Ru) with the cumulative energy of RIC and HVDM at 2-meters depth is as shown in Figure 11. In the graph, energy was assumed to accumulate after each drop. The excess pore water pressure was calculated by reading pore water pressure from the piezometer after each drop and then subtracted it with initial in-situ pore water pressure to obtain the Ru value. The Ru formula is shown in equation (1), which is defined as the ratio of excess pore water pressure to the initial effective vertical stress. A higher value of Ru (0.9 or greater) will result in boiling or soil liquefaction. From Figure 11, for RIC, Ru at 2 meters depth was almost 1.0 and therefore risked boiling. HDVM had a similar problem with RIC at 2 meters depth where the Ru value was above 1.0.

$$\mathrm{Ru} = \Delta u / \sigma'_v \tag{1}$$

Where Δu is excess pore water pressure, and σ'_v is initial effective vertical pressure.

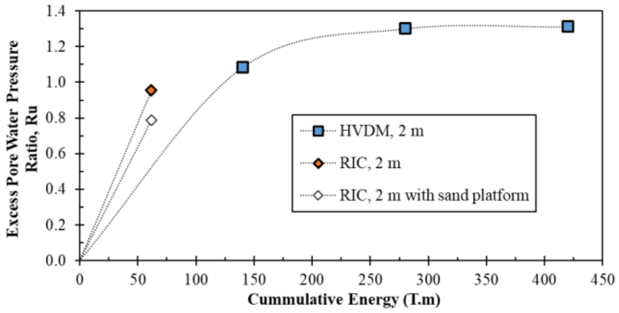


Figure 11 Excess pore water pressure ratio (Ru) vs cumulative energy (T.m) of RIC and HVDM

The boiling occurrences may create a problem with the improvement effectiveness when the blow is applied to the soil surface. The impact influence might decrease severely, and the probability of the RIC machine sinking into the soil body increases. For the solution, a theoretical 0.5-meter-thick working platform of a sand layer was added to the initial effective stress (σ'_v) in the Ru calculation of the RIC and the result is shown in Figure 11. The Ru for 2 meters depth decreased approximately 10-15%. This result indicated that an additional soil layer as a working platform is necessary to reduce Ru value to prevent soil boiling and to increase the compaction effectiveness and safety of the RIC machine and HVDM machine.

3.3 Construction period of RIC and HVDM

The dissipation of excess pore water ratio curve can be used to determine the waiting period for the RIC method and the HVDM method, which has an impact on reducing the safety limit of machine that operates on the field from both methods. In Figure 12, it is shown that at 2 meters depth for the RIC method, which used lower compaction energy than HVDM method, the excess pore water pressure ratio needs approximately 72 hours (3 days) to reach below the value of 0.2. For the HVDM method, the excess pore water pressure ratio is reduced relatively slowly at 120 hours and needs approximately 168 hours (7 days) to reach below the value of 0.2. However, the Ru drops relatively fast from 1 to 0.75 for the HVDM method during the initial hour that showed the excess pore water pressure may dissipate with the help of the vacuum pump.

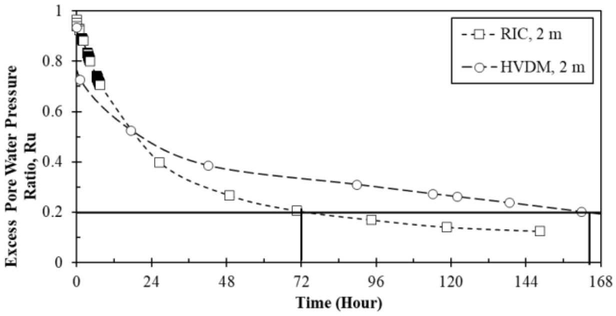


Figure 12 The dissipation over time of excess pore water pressure ratio (Ru) for RIC and HVDM at 2 meters depth

The waiting period is essential to prevent the machines from sinking into the soil body and many other risks that may damage the heavy machines. Therefore, the machine will be able to continue to the next stage of improvement safely. Ru value 0.2 was selected as the approximate limit because the vertical value of Ru curve for both methods start to decrease slowly when roughly approaching the mentioned Ru value. Based on the previous explanation, it can be concluded that the waiting period for the RIC should be at least 3 days and at least 5-7 days for the HVDM. These waiting periods may change according to cumulative energy of compaction and soil conditions such as the soil type uniformity in the subsoil, percentage of fines content present in soil and groundwater table depth.

3.4 Field test results

The CPT and SASW test results are shown in Figure 13 and Figure 14. It can be observed that below the existing clay layer at the depth of 3 to 4 meters, the improvement was almost negligible using the RIC method, while a small improvement was observed from the HVDM method.

For RIC Trial area 2, the CPT result shows an effective improvement from 0 to 2.5 meters in depth and shows some improvement at 4 to 5 meters in depth. The Vs profile shows an increase in the top layer (0-1.5 meters) and a slight increase for the soil layers up to 7 meters.

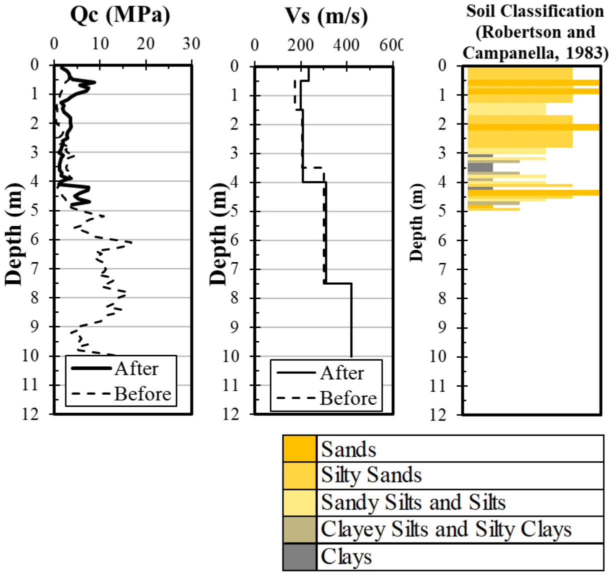


Figure 13 CPT and SASW result of RIC Trial Area 2

For the HVDM trial area, the CPT data from trial area 71is chosen to represent the soil improvement for the HVDM method which shows an effective improvement from 1-meter depth down to 3.5-meters depth and some slight improvement up to 5.5 meters depth. In this trial area, the SASW test was only performed after the improvement was finished. Thus, q_c of CPT data in this trial area was converted to Vs to be compared before and after the improvement in terms of Vs. CPT data can be converted to Vs by using an equation for Pleistocene incoherent soils (Madiai and Simoni, 2004; Amoroso, 2013) with the following formula:

$$Vs = 172. q_c^{0.35} . f_s^{-0.05} \tag{3}$$

Where q_c is tip resistance in mPa; f_s is sleeve friction in MPa.

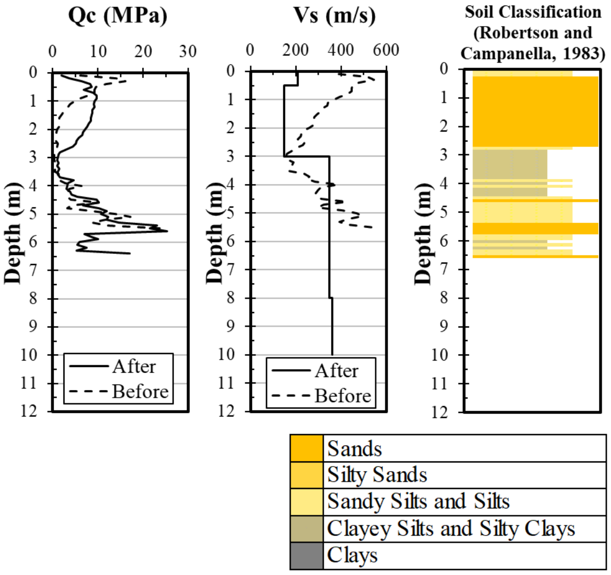


Figure 14 CPT and SASW result of HVDM Trial Area 71

It can be concluded that the HVDM method can improve the soil with a high fines content at such depth. At the depth of 2 meters, tip resistance (q_c) shows the value of 5-7 MPa for RIC method and 10 MPa for HVDM method. It shows at such depth the soil was denser at HVDM trial area that could be the reason of slower dissipation rate at 2 meters for HVDM method compared with RIC method.

4. CONCLUSION

Based on the data that has been shown, it can be concluded that:

1. From the excess pore water pressure, the liquefaction potential can be observed in the form of excess pore water pressure ratio. The Liquefaction potential is present at the depth of 2 meters which is the top layer, since the upper layer of the soil will experience much more of the compaction energy than lower layer of the soil. By putting a 0.5-meter-thick sand platform on top of the original soil layer, the liquefaction potential could be decreased by 10-15%.
2. The waiting period is necessary for the machine to be able to continue to the next stage of the improvement safely so that the pore water pressure dissipates leading to the reduction of Ru value to a certain number. The required waiting period for the RIC method and HVDM method are 3 days and 5-7 days respectively. It is also worth mentioning that the differences in compaction cumulative energy and soil condition, such as fines content, might also affect the waiting time.
3. The field test result shows that the HVDM method was able to effectively improve soil from ground surface up to 3.5 meters with the maximum improvement depth of 5.5 meters, while the RIC method was only able to improve soil effectively from the ground surface up to 2.5 meters with the maximum improvement depth of 5 meters. The existing high fine-grained soil may reduce the effectiveness of the improvement in some parts of soil layers.

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6. REFERENCES

Alditra, D. F., Chaiprakaikeow, S., & Soralump, S. (2020). Study of VCM Improved Soft Soil Properties Using Non-Destructive and Destructive Techniques. *Geosciences*, *10*(8), 300.

Amoroso, S. (2013). Prediction of the Shear Wave Velocity V S from CPT and DMT. Paper presented at the Proceedings of the 5th International Young Geotechnical Engineers' Conference. doi:https://doi.org/10.3233/978-1-61499-297-4-473

Avalle, D., Scott, B., and Bouazza, A. (2019). Comparing the relative merits of dynamic compaction, rapid impact compaction and impact rolling. Paper presented at the 13th Australia New Zealand Conference on Geomechanics, Sydney, Australia.

Barus, R.M.N.; Jotisankasa, A.; Chaiprakaikeow, S.; Sawangsuriya, A. Laboratory and Field Evaluation of Modulus-Suction-Moisture Relationship for a Silty Sand Subgrade. *Transp. Geotech.* **2019**, *19*, 126–134.

Chen, K., Liu, H., and Lo, R. (1998). Treatment of Soft Soils by Dynamic Compaction. Paper presented at the Fourth international conference on case histories in geotechnical engineering (1998: March 8-12; St. Louis, Missouri).

Cheng, S. H., Chen, S. S., & Ge, L. (2021). Method of estimating the effective zone induced by rapid impact compaction. *Scientific Reports*, *11*(1), 1-16.

Chow, Y., Yong, D., Yong, K., and Lee, S. (1992). Dynamic compaction of loose sand deposits. Soils and foundations, 32(4), 93-106. doi:https://doi.org/10.3208/sandf1972.32.4_93

Ismail, M.; Samsudin, A.; Rafek, A.; Nayan, K. Road Pavement Stiffness Determination Using SASW Method. *Unimas E J. Civ. Eng.* **2012**, *3*, 9–16.

Ji-hong, C. (2014). HVDM Method Application in a Sandy Clay Ground. Proceedings of Softsoils, 21(23rd).

Joh, S.H.; Stokoe, K.H. *Advances in Interpretation and Analysis Techniques for Spectral-Analysis-of-Surface-Waves (SASW) Measurements*; Offshore Technology Research Center: College Station, TX, USA, 1997.

Liang, R. and Xu, S. (2010). Innovative Soft Clay Improvements Using Vacuum and Dynamic Compaction. Paper presented at the Proc. Indian Geotechnical Conference-2010.

Madiai, C. and Simoni, G. (2004). Shear wave velocity-penetration resistance correlation for Holocene and Pleistocene soils of an area in central Italy. Paper presented at the Proceedings, 2th International Conference on Geotechnical Site Characterization (ISC'2), Porto, Portugal.

Mase, L.Z.; Likitlersuang, S.; Tobita, T.; Chaiprakaikeow, S.; Soralump, S. Local Site Investigation of Liquefied Soils Caused by Earthquake in Northern Thailand. *J. Earthq. Eng.* **2018**, *22*, 1–24.

Menard, L. and Broise, Y. (1975). Theoretical and practical aspect of dynamic consolidation. Geotechnique, 25(1), 3-18. doi:https://doi.org/10.1680/geot.1975.25.1.3

Nashed, R., Thevanayagam, S., and Martin, G. (2009). Dynamic compaction of saturated sands and silty sands: results. Proceedings of the Institution of Civil Engineers-Ground Improvement, 162(2), 69-79. doi:https://doi.org/10.1680/grim.2009.162.2.69

Nazarian, S.; Stokoe, I.; Kenneth, H.; Briggs, R.C.; Rogers, R. Determination of Pavement Layer Thicknesses and Moduli by SASW Method. *Transp. Res. Rec.* **1988**, *1196*, 133–150.

Nazarian, S.; Stokoe, K.H. *In Situ Determination of Elastic Moduli of Pavement Systems by Spectral-Analysis-of-Surface-Waves*

Method: Practical Aspects; Research report 368-1F; The University of Texas at Austin: Austin, TX, USA, 1985; 161p.

Robertson, P.K. and Campanella, R. (1983). Interpretation of cone penetration tests. Part I: Sand. Canadian geotechnical journal, 20(4), 718-733. doi:https://doi.org/10.1139/t83-078

Rosyidi, S. A., Nayan, K. A. M., Taha, M. R., & Ismail, A. (2006). Estimating G-Max & field CBR of soil subgrade using a seismic method. eJ. Nondestr. Test., 11

Shrestha, A.; Jotisankasa, A.; Chaiprakaikeow, S.; Pramusandi, S.; Soralump, S.; Nishimura, S. Determining Shrinkage Cracks Based on the Small-Strain Shear Modulus–Suction Relationship. Geosciences 2019, 9, 362.

Soralump, S., Panthi, K., and Prempramote, S., "Assessment of the upstream slope failure of a dam due to repeated cyclic drawdown," Soils and Foundations, vol. 61, no. 5, pp. 1386-1398, 2021.

Tarawneh, B. and Matraji, M. (2014). Ground improvement using rapid impact compaction: case study in Dubai. Građevinar, 66(11.), 1007-1014. doi:https://doi.org/10.14256/JCE.1083.2014

Yao, K., Rong, Y., Yao, Z., Shi, C., Yang, C., Chen, L., Zhang, B., et al. (2022). Effect of water level on dynamic compaction in silty ground of Yellow River alluvial plain. Arabian Journal of Geosciences, 15(1), 1-16. doi:https://doi.org/10.1007/s12517-021-09426-9

Zhong, R., & Haryono, F. (2019). Case studies of an innovative ground improvement method-High vacuum densification method. Paper presented at the 13th Australia - New Zealand Conference on Geomechanics