Dynamic Compaction at New Yogyakarta International Airport for Liquefaction Mitigation

R.R. Setiaji[1], A.M. Hakim[2], F.H. Adinda[3], and K.M. Abuhuroyroh[4]
[1]BUT Menard Geosystems, Jakarta, Indonesia
[2]BUT Menard Geosystems, Jakarta, Indonesia
[3]BUT Menard Geosystems, Jakarta, Indonesia
[3]Independent Technical Expert for Ground Improvement, Bandung, Indonesia
E-mail: ryan.setiaji@menard-asia.com, abi.hakim@menard-asia.com, febrini.hartianty@menard-asia.com, roy.yusfi@gmail.com

ABSTRACT: In high seismic areas, liquefaction is a major issue when a designer dealing with loose sand below groud water level and low fines content. To solve this problem, densification by means of compaction is the most simple approach to be accomplished. Dynamic compaction technique by repeated impacts of a heavy weight on to the soil can successfully densify the soil and thus, dismiss the liquefaction issue. The project case described in this paper is the development of New Yogyakarta International Airport, which is located in Kulon Progo, Indonesia. It is located in high seismic area with Peak Ground Acceleration (PGA) of 0.4g. Sand with fines content ranging from 0.25% to 17.23% was found all over the project site. Furthermore, the design criteria was stated to mitigate the liquefaction during the designed earthquake. The evaluation of project achievement to mitigate the liquefaction was performed based on In-Situ Testing Parameter; in this case is Cone Penetration Test. This paper presents a brief discussion of the entire project case which is using Dynamic Compaction method to mitigate the liquefaction issue.

Keywords: Ground improvement, dynamic compaction, liquefaction mitigation, high seismic area, cone penetration test, new airport.

1. INTRODUCTION

Liquefaction has been a major issue in geotechnical engineering of the countries within seismic belt. Physically, loose sand with low fines content below groud water level tends to densify subjected to loading; the excess pore water pressure would barely build up due to its eminent permeability. Nevertheless, when the loading is occured immensely and repeatedly in very quick fashion e.g. earthquake loading, the excess pore water pressure is developed much faster than it takes to dissipate untill an extent that the sand loose of all its strength, liquefaction. When it happens, the upper structure is no longer supported and building collapse is inevitable. Evidence of liquefaction has been prevalent in historic earthquakes (Obermeier, 1998; Tuttle et al., 2000; Schneider et al., 2001). The most susceptible to liquefaction is sediments, including saturated Holocene to late Pleistocene age deposits, river channel and flood plain alluvium, aeolian deposits, and poorly compacted fills (Youd, 1991; Krinitzsky and Hynes, 2002).

Liquefaction can be prevented by densification the existing sand deposits or reclaimed sand using Dynamic Compaction method. By performing repeated impact of a heavy mass on to the soil, Dynamic Compaction produces sands with denser state. The denser sands become, the less liquefaction potential would be. As one of the ground improvement technique, this method has been recognized as very effective in densification of loose soil within a variety of depth (Menard and Broise 1975, Lukas 1980, Chow et al. 1992). In landfill areas, it has also been successfully utilized for numerous purposes (Mayne et al. 1984).

This paper presents experiences gained from case history where Dynamic Compaction technique was adopted for the densification of existing natural sand deposits. The project case history was a coastal where the soil investigation carried out before the work shows that the site was composed of natural sand deposits with low fines content. Based on earthquake map of Indonesia, the project was located in high seismic zone. The liquefaction sucseptibility of the site was obvious, hence the liquefaction criteria was developed as a main objective of ground improvement works. The calibration test was performed prior to the densification works to determine the optimum energy required. The production and post-production monitoring and evaluation was also performed. It shows that the Dynamic Compaction is successfully met the required design criteria. The works was completed with high productivity resulting a short and compact working period.

2. DYNAMIC COMPACTION

The basic principle of the dynamic compaction is transmission of high energy impacts to loose soils which has originally high compressibility, low bearing capacity, and liquefaction potentials with the purpose of substantially enhance their characteristic with depths. The mechanism of dynamic compaction is repeatedly dropping a heaving pounder (15-20 tons) from a height (5 to 20 m) on to soil surface. The compaction is normally carried out in few phases, the first phase is generally higher energy to compact the deepest treatment zone, and subsequent phase will be carried out with lesser energy and finally ironing phase that consists of overlapping pounding with lower drop height to treat the upper 2-3 m. It is common that the top 0.5 m is not well compacted due to leveling activities after ironing phase. This layer shall be compacted by roller compactor to compact the surface layer. The high energy impacts will generate compression wave that build up porewater pressure and dislocate the soil particles. Shear wave and Rayleigh wave would rearrange the soil particles into denser form as shown in Figure 1.

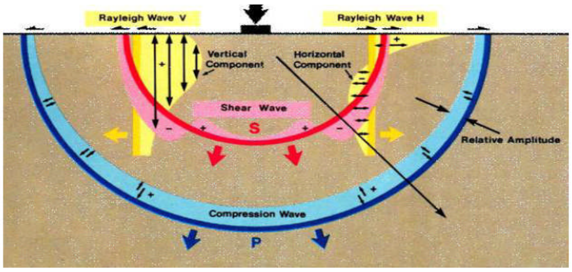

Figure 1 Waves generate by Dynamic Compaction

The degree of improvement is a function of the applied energy where it depends on tamper weight, drop heights, number of blows and grid pattern. In term of depth of improvement, it depends on tamper weight and drop height. The empirical correlation below could be used for preliminary design.

$$D = n\,(W.H)^{0.5} \qquad (1)$$

D = depth of Improvement
n = empirical coefficient
W = weight of pounder
H = drop height

The emprical coefficient n is the range of 0.3 to 1.0 depending on soil types, efficiency of the drop mechanism and existance of hard layer or energy absorbing layer. Table 1 shows the recomended values for n extracted from the past experience and record.

According to FHWA-SA-95-037 (1995), the applied energy is generally given as the average energy applied over the entire area.

$$AE = \frac{W \cdot H \cdot N \cdot P}{G^2} \tag{2}$$

AE = applied energy
N = number of drops at specific drop point
W = mass of tamper in tons
H = drop height in meters
P = number of passes
G = grid spacing in meters

More recently, Varaksin and Racinais (2009) have proposed Eq. (3) which considers the degree of improvement as a function of depth.

$$f(z) = \frac{f_2 - f_1}{D^2}\,(z - NGL)^2 + f_1 \tag{3}$$

where,
f(z) = improvement ratio at elevation (z)
z = elevation in meters
NGL = natural ground level
D = depth of influence of dynamic compaction
f_1 = maximum improvement ratio at surface
f_2 = improvement ratio at maximum depth of influence

Table 1 Recommended Values for n

Source	n-values	Soil Type
Menard & Broise (1975)	1.0	All soils
Leonards, Cutter & Holtz (1980)	0.5	-
Smoltczyk (1983)	0.5	Soil with unstable structure
	0.67	Silts and sands
	1.0	Pure frictional soils
Lukas (1980)	0.65-0.8	-
Mayne, Jones, & Dumas (1984)	0.3-0.8	-
Gambin (1985)	0.5-1.0	-
Qian (1987)	0.65	Fine sand
	0.66	Soft clay
	0.55	Loess
Vsn Impe (1989)	0.65	Silty sand
	0.35	Municipal waste
	0.5	Clayey sand
Yee, Setiawan & Baxter (1998)	0.5	Calcareous sand/coral sand
Faisal, Yee & Varaksin (1997)	0.33-0.59	Municipal waste

3. CASE STUDY: NEW YOGYAKARTA INTERNATIONAL AIRPORT

3.1 Project Description

The project was located in Kulon Progo Regency, Yogyakarta Province. The exact location can refer to Figure 2.

The total of treatment area, which is composed of one runway, two rapid exit taxiways, two holding bay, one paralel taxiway, three exit taxiways, one taxiway apron, and one apron, was approximately 900,000 m^2. Figure 3 shows the ground improvement works area.

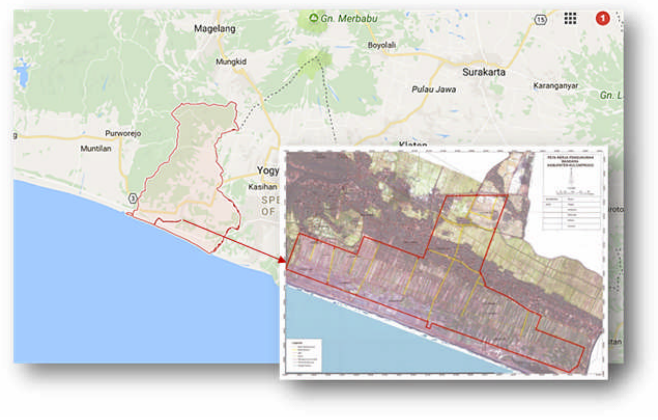

Figure 2 Project location

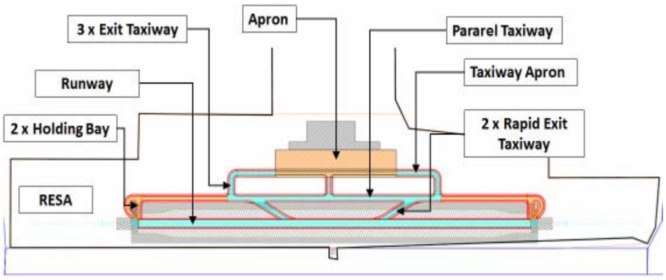

Figure 3 Project ground improvement area

3.2 Design Criteria – Liquefaction Mitigation

The project was located within high seismic area as shown in Indonesia seismic map, Figure 4, with Peak Ground Acceleration (PGA) value of 0.4g. The design criteria was to mitigate the liquefaction, with moment magnitude, Mw, of 7.8 and safety factor against liquefaction of 1.2.

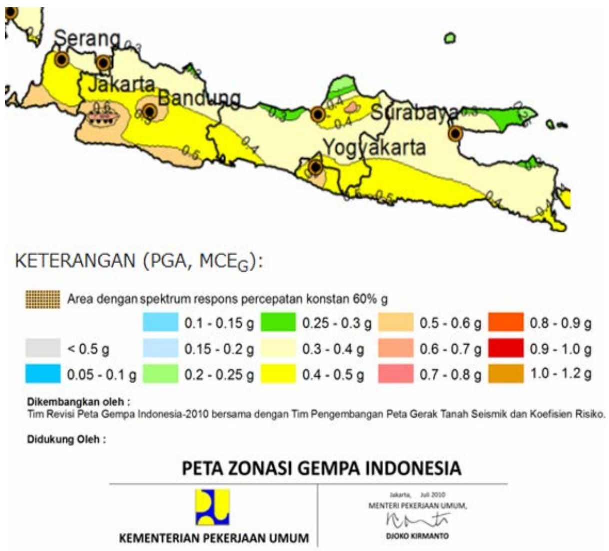

Figure 4 Indonesia seismic map 2012

In regard to the seismic data, the quick acceptance criteria was then developed to evaluate the compaction results using simplified procedure of 1996 NCEER and 1998 NCEER/NSF (Figure 5). It was presented in q_c value versus depth, as the post-in-situ-test was using Cone Penetration Test. The quick acceptance criteria is presented in Figure 6.

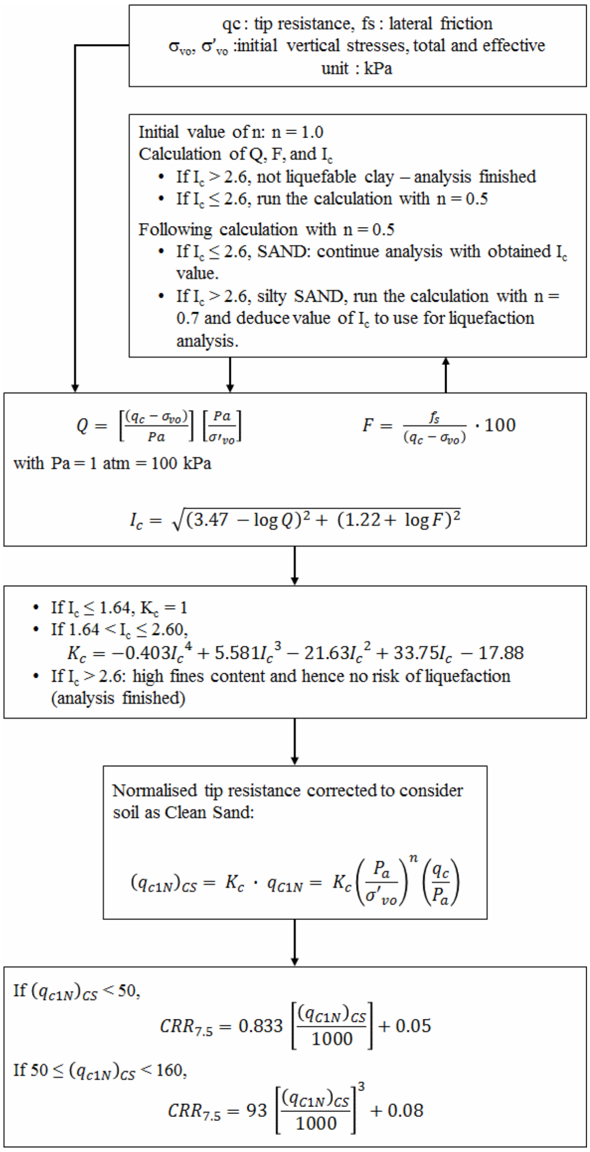

Figure 5 Evaluation risk of liquefaction procedure based on simplified NCEER 1998

3.3 Geotechnical Condition

To understand comprehensively about the soil condition, it is better to identify about the history of soil formation specifically at project site. The site was located within the watershed of rivers originally coming from the Merapi Volcano. The nature of sand in project sand is predominantly black colored. This kind of sand is well-known in Southern Java as Iron Sand, which is abounding of iron mineral. Iron sand is commonly found along the coast, formed by the process of razing the origin rocks from Merapi Volcano by weather and surface water, which are then transported and deposited along the coast. Waves of sea with a certain energy sorted and accumulated the sediment into iron sand deposit. Figure 7 presents the illustration of the sedimentation process. Figure 8 shows the site condition prior to dynamic compaction work.

A robust geotechnical investigation was performed by the consultant prior to project design and execution. The in-situ test consisting of 84 boreholes were combined with laboratory test. The plot of N-SPT data from boreholes is presented in Figure 9.

Generally, the soil condition was sand. Based on boring logs and N-SPT data, it was found that the top four meters is composed of loose to medium dense sand (N-SPT: 3 – 25 blows/30 cm). Then, the next two meters below the top layer the sand condition was in denser state (N-SPT: 15 – 50). At eight-meter depth and below, the sand was found in very dense form (N-SPT >50).

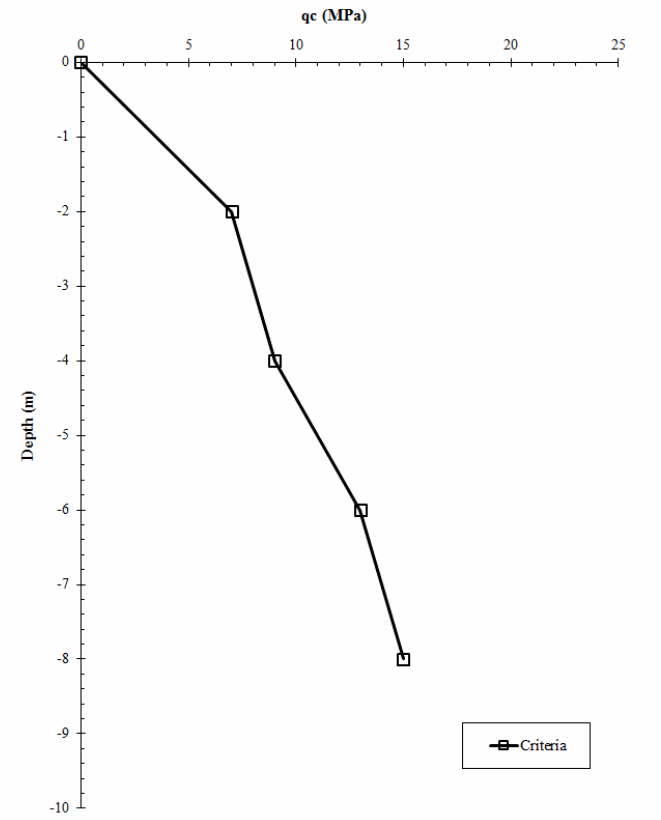

Figure 6 Quick acceptance criteria

Based on the laboratory result, it was found that the sand throughout the project site having fines content ranging from 0.25% to 17.23%. Hence, it can be simplified that the sand was compounded of clean sand to silty sand.

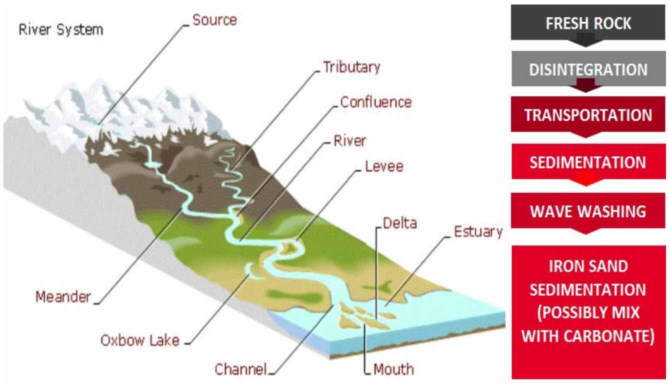

Figure 7 Water flow in river system and sedimentation process

Figure 8 Site condition prior to ground improvement work

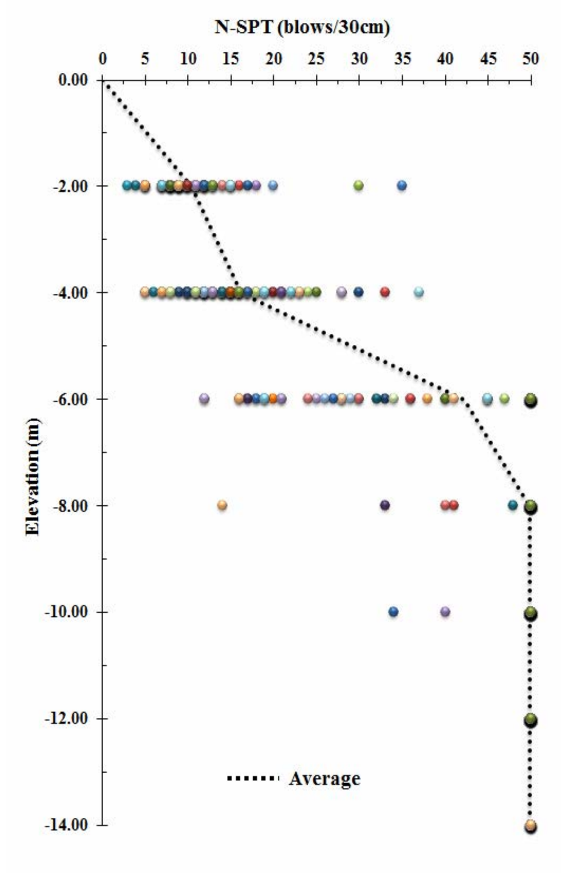

Figure 9 N-SPT summary graph

3.4 Dynamic Compaction Works

Dynamic compaction (DC) method was chosen based on the required treatment depth that is clearly suitable, as the maximum treatment depth for DC is 10 meters. The fines content was also properly fit below the maximum fines content for DC work which is 20%. Environmentally, the site was located in green area and almost no critical structure nearby the site so that the vibration induced by the works would not be an issue.

As automatic free fall method was used (Figure 10), the properties of pounder used for the DC work was as follows:

- Shape : Circular
- Diameter : 2.20 m
- Height : 1.25 m
- Weight : 17 ton

Figure 10 Dynamic compaction rig and pounder on site

3.4.1 Field Calibration Test

A full scale of calibration test was performed on site. An area of 2 x 20 m x 20 m (total area of = 800m^2) was selected for field calibration test. This test incorporated heave and penetration test, pre- and post- cone penetration test (CPT). The objective of this calibration test was to determine the optimal compaction energy, which is composing of:

- Number of blows
- Drop height
- Spacing and grid pattern
- Magnitude of heaving

The purpose of heave and penetration test (HPT) was to determine the optimal energy (number of blows) required for the compaction works. The test involved calculating the penetration depth, heave volume, print volume and the effective volume (i.e., print volume minus heave volume). The data was recorded and plotted into graph as shown in Figure 11 and Figure 12.

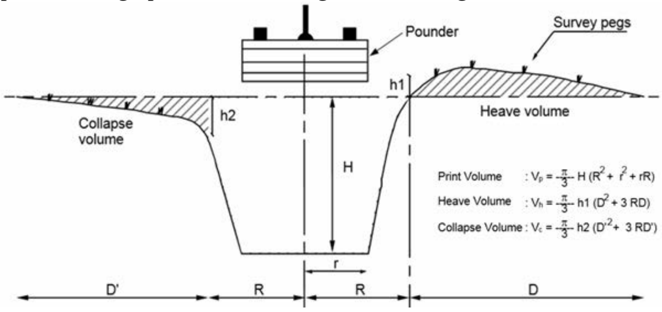

Figure 11 Typical cross section due to pounder compaction

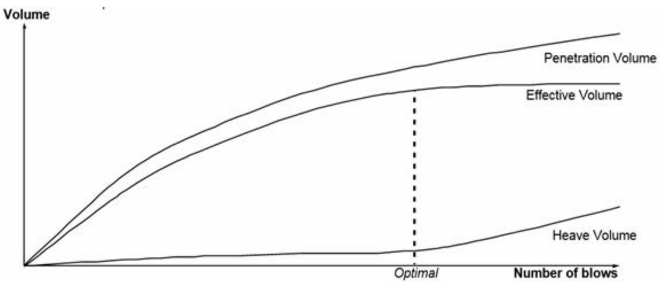

Figure 11 Typical graph to analysed HPT result

The drop height was selected 15 m. Based on heave and penetration test, the result was analysed and evaluate to obtain the optimum production parameter (Figure 12). As shown in Figure 13, the effective volume is started to change in shape at blows 4 and 5, indicating that the effective blows to be use is either 4 or 5. Accordingly, the calibration test was performed using 4 blows at Zone A and 5 blows at Zone B (Figure 14).

After the calibration test was carried out, several post-CPT test was performed as shown in the Figure 14. The result was showing that both zones achieved the design criteria. As conclusion, DC work was comprised of one phase with 5 m spacing, square grid pattern, and 4 number of blows.

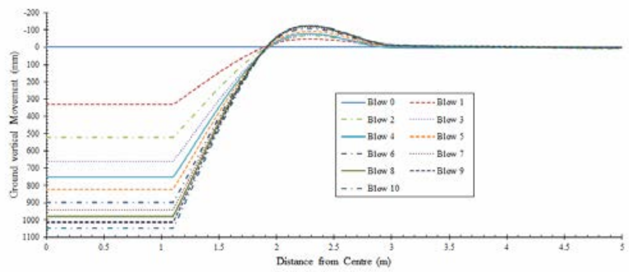

Figure 12 Ground settlement and heave with distance

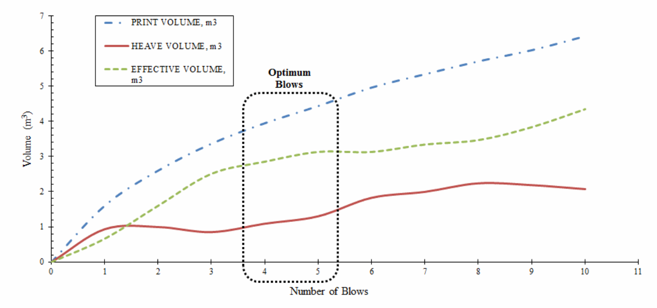

Figure 13 Volume vs number of blows

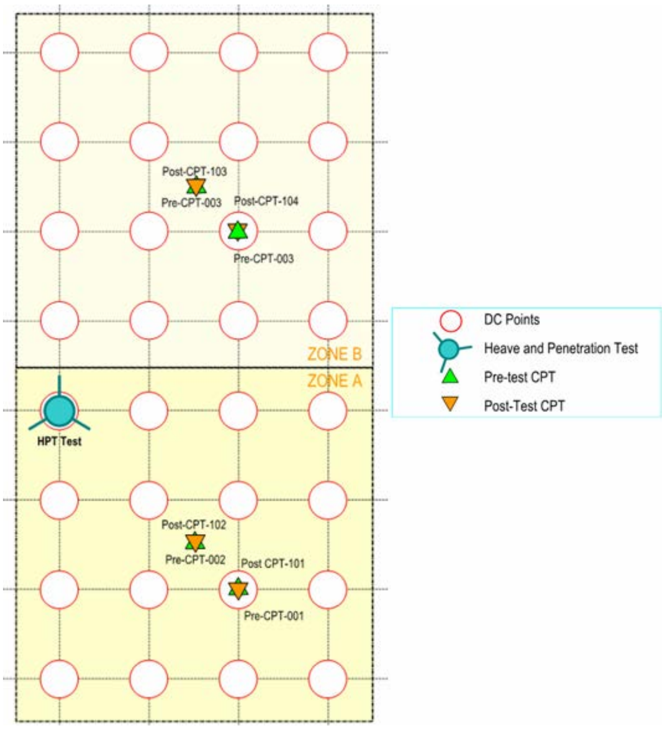

Figure 14 Schematic plan of field calibration test

3.4.2 Construction Performance

Menard has maintained the working capacity and quality through the time since the beginning. The highest production capacity at the site was 13,000 m^2/day and the average was about 8,000 m^2/day. The total area of 40 Ha was finished completely within 3 months. Total of volume sand treated by the dynamic compaction works was approximately 2,500,000 m^3. Figure 15 presents the documentation during compaction work on site.

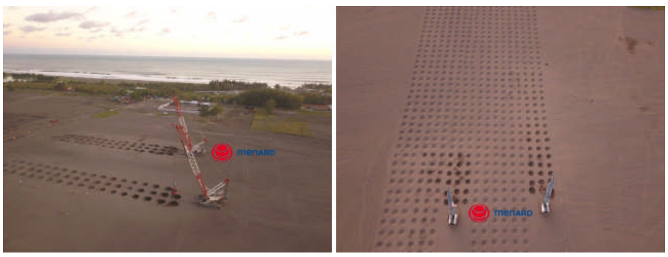

Figure 15 Dynamic compaction work on site

3.4.3. Quality Assessment and Quality Control

As part of quality assessment and quality control, cone penetration test was carried out for every 50 m x 50 m (2,500 m^2) of treatment area throughout the project site for both pre- and post- ground improvement work. By using this scheme, the evaluation of the work was more comprehensive and the data was more representative

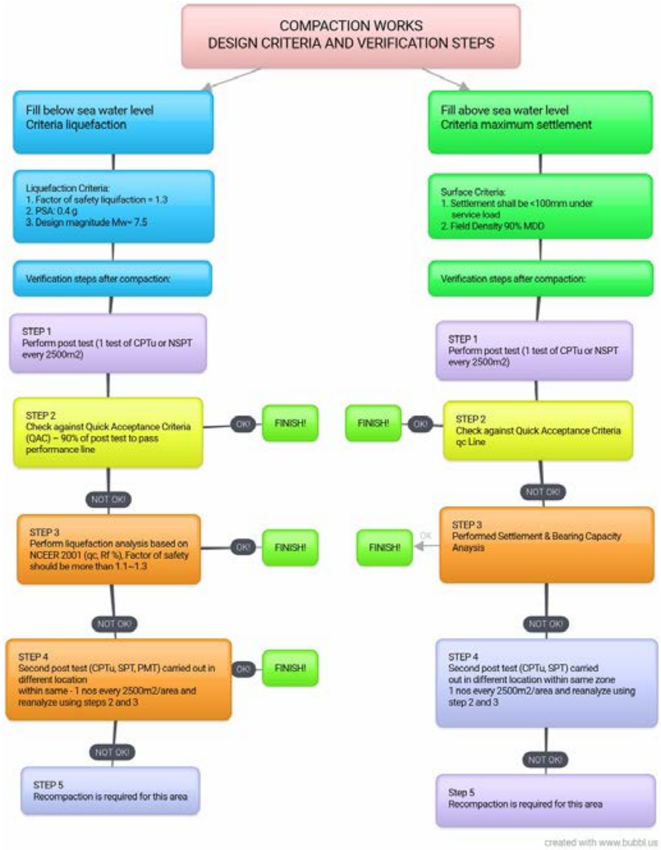

Figure 16 Work acceptance flowchart

all over the site. The work acceptance flow chart was also estabilished as presented in Figure 16. The first-rate CPT rig with capacity of 20 ton and auto-generated CPT result was used to carry out the tests (Figure 17).

Figure 17 CPT rig with 20 ton capacity and auto-generated result

3.5 Result and Evaluation

As an evaluation, the pre-test CPT and post-test CPT result are compared. The post tests result are showing that the dynamic compaction work is resulting the post CPT qc value above the qc value of quick acceptance criteria. The sample of comparison between pre-CPT and post-CPT are presented in Figure 18.

The improvement ratio is also analysed to obtain more big picture of the result of dynamic compaction work at the project site. The graph is showing that the improvement ratio is vary to 6.5 depth

(Figure 19). The 0.5 m thick layer on top is loosened certainly due to the mechanism of heaving. The improvement ratio increase until reach the peak at 3.2 m, so-called optimal depth, then decrease as the depth getting deepened and the existing qc value getting higher. The improvement ratio is insignificant after the existing qc value higher than 25 MPa. The average improvement ratio is approximately 1.8.

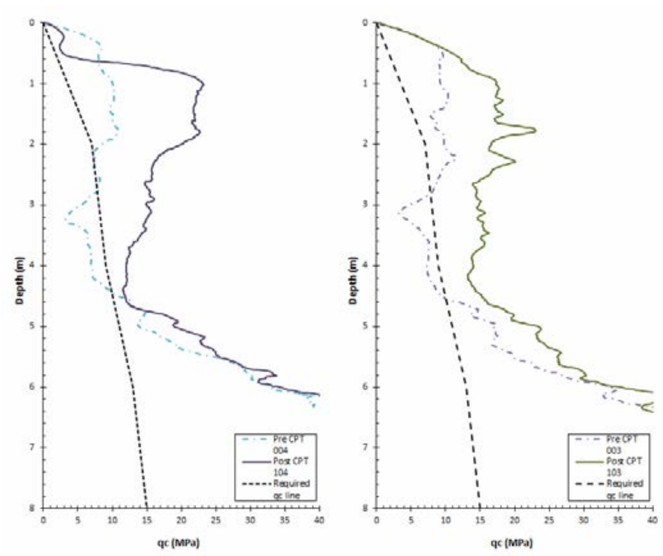

Figure 18 Comparison of pre-test and post-test CPT

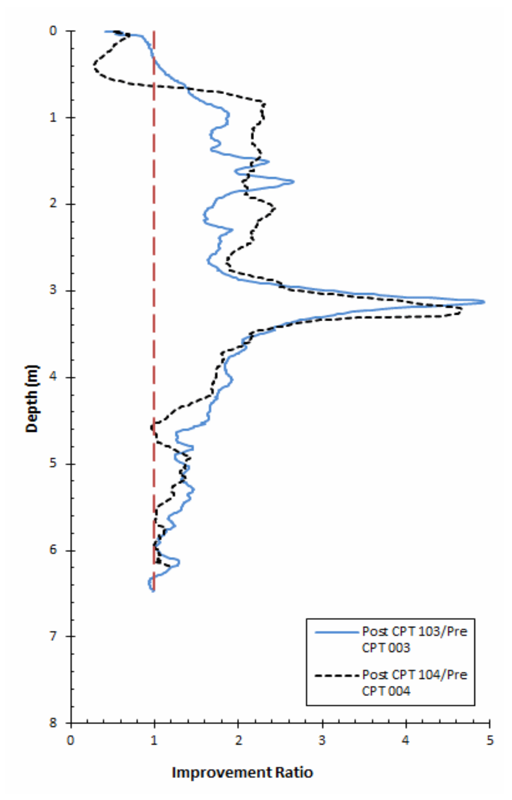

Figure 19 Improvement ratio of dynamic compaction works

4. CONCLUSION

Dynamic compaction has demonstrated to be a successful technique to densify and mitigate the liquefaction issue in high seismic area. This paper has highlighted the effective implementation of dynamic compaction to treat liquefiable ground condition for new international airport in Indonesia with compaction area of approximately 900,000 m^2. The calibration test is required to be carried out prior to dynamic compaction work. Proper quality assessment and quality control is essential to maintain the accurate result and evaluation.

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