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Determination of Downhole Dynamic Compaction Parameters using Finite Element Analysis

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ABSTRACT Downhole dynamic compaction (DDC) has been commonly used in China to stabilize collapsible soil through the application of construction and demolition waste material (CDW). DDC basically forms a column inside the soil stratum which is similar to a stone column except DDC materials are put in sequence and then compacted by using DDC hammer. Due to its attractive features such as its big diameter, feasibility of using oversized material particles, rapid and simple construction technique, it is used as one of the ground improvement methods for an airport project in Indonesia. Despite of all the advantages provided by DDC, it is difficult to obtain DDC parameters from laboratory tests as it is difficult to replicate the compaction effort induced by the DDC hammer and laboratory tests are not commonly employed for oversized materials. Hence, alternative method is required to evaluate DDC parameters. In this study, static load test is conducted to determine load-deformation curve of the DDC pile. Soil parameters are first determined through soil test data such as standard penetration test (SPT), laboratory test and also pressure meter tests. Correlation between pressure meter tests and SPT test result is also carried in order to interpret the soil parameter at the site. Axisymmetric finite element analysis is then carried by using MIDAS GTS NX in order to back analyses DDC parameters by matching the simulation curve with load settlement curve of the DDC. In this paper, it is shown that back analysis using hardening soil model for DDC material can be used to match simulation curve with the load-deformation curve.

KEYWORDS DDC; Volcanic Soil; Finite Element Method; Back Analysis; Static Load Test.

1 INTRODUCTION

Downhole dynamic compaction (DDC) has been used to stabilize collapsible soil (i.e. loess) in China (Feng *et al*., 2015). The DDC pile can be constructed using the pre-boring method which involves pre-boring a hole before filling it with DDC materials or through self-tamping method which focuses on dropping a hammer (in Figure 1) on the pile location until the hole reaches the desired depth as illustrated in Figure 2. This self-tamping method is commonly used due to its ability to cause dynamic lateral stress which densifies the surrounding soil, especially when the soil is loess collapsible soil (Feng *et al*., 2015). Meanwhile, the pre-boring method is mostly used when the self-tamping method is not usable (usually due to a very hard soil layer).

Figure 1. The DDC Hammer (9.7-ton weight)

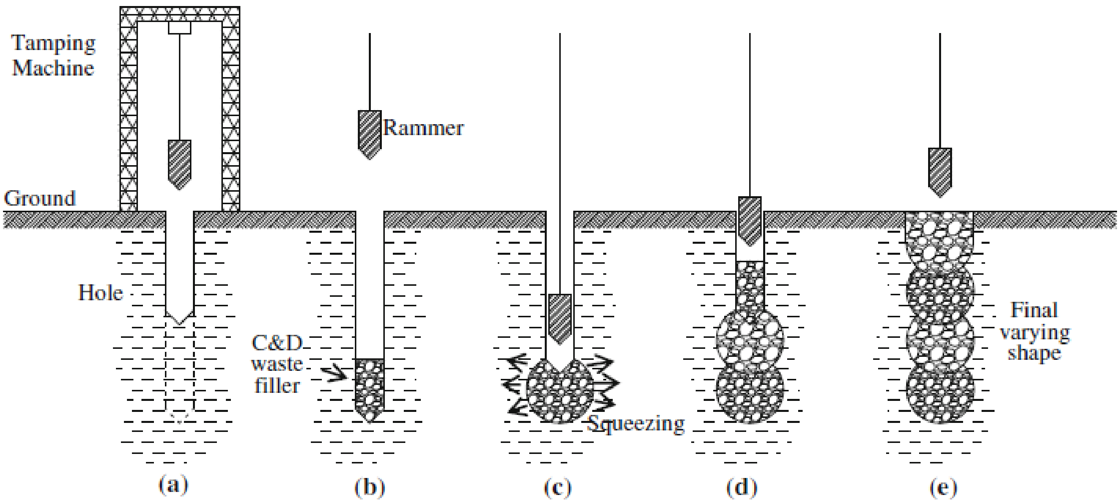


Figure 2. Self-tamping procedures of DDC (Feng *et al*., 2015)

The DDC material can be poured into the hole after it has been created and this is followed by the lifting of the hammer to a specific height and dropping for a specific number of times as shown in Figure 2. It is important to note that construction and demolition waste materials (CDW) such as concrete, brick, and rocks are usually used as the DDC material in order to ensure an ecofriendly environment, specifically in countries with rapid development such as China where more than 100 Million tons of CDW are produced every year (Zhang *et al*., 2012).

The DDC was used in this study as one of the ground improvement methods. This led to the application of materials mined from the site which are referred to as surface mining (SM) and rock excavation (RE) instead of CDW. It is important to note that the particle size of these materials is up to 200 mm as indicated in Figure 3.

The DDC parameters need to be determined because the piles are usually used to reduce the occurrence of possible settlements during the airport service period. However, it is difficult to use both laboratory and in situ tests to determine these parameters due to the existence of oversize materials. This led to the application of alternative methods such as back analysis through the finite element method to evaluate the parameters.

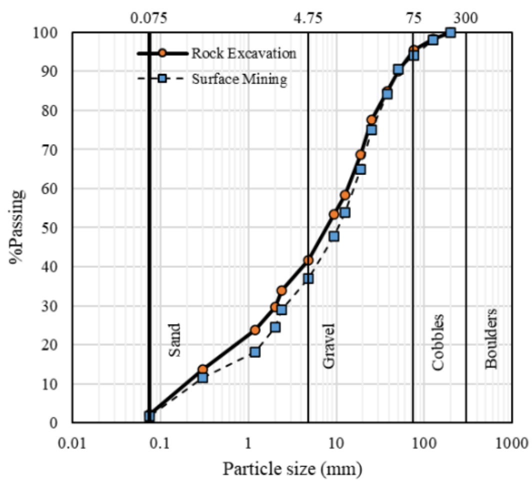


Figure 3. Grain size distribution of DDC Material

(a) (b)

Figure 4. Photo of DDC tamping procedure (a). Lifting the DDC hammer and (b). Dropping the DDC hammer

Figure 5. Geological Map of the Madiun Quadrangle and Google Earth TM

2 METHODS

A new airport was recently constructed in Dhoho, Indonesia. The geological formation of the project was observed to be volcanic rocks of the Quarternary Pleistocene Period (Pawonsewu Morphocet – Qp) which consists of volcanic breccia with pyroxene andesite fragments, tuff, agglomerate, and pyroxene andesite lava as indicated in the geological map presented in Figure 5. An important observation in volcanic or residual soil is that it does not undergo consolidation such as sedimentary soil. This was confirmed by Wesley (2010) that pre-consolidation pressure and over-consolidation ratio (OCR) terms are not appropriate for residual soil because it does not experience the sedimentation process. This led to the formulation of yield stress or yield pressure to replace the pre-consolidation pressure. This yield stress does not only represent the stress history but also the diagenesis, bonding, fabric, and intrinsic structural alterations which can possibly occur during the weathering process (Mayne, 2013; Mayne, 2014; Wesley, 2010). This means several terms such as apparent OCR (Wesley, 2010) and yield stress ratio YSR (Mayne, 2014) are more appropriate than OCR for residual soil.

The correlation between SPT-N and YSR was determined from an oedometer test by Pangaribuan (2021), as presented in Figure 6. Pangaribuan (2021) also shows that the YSR can be appropriately estimated using Mayne and Kemper's (1988) equations as follows:

$$YSR = 0.193\left(\frac{N}{\sigma_v'(MPa)}\right)^{0.689} \tag{1a}$$

$$YSR = 22.519\left(\frac{N}{\sigma_v'(kPa)}\right)^{0.689} \tag{1b}$$

Where σ_v’ in Equation (1) is vertical effective stress in MPa. It is important to note that this equation can overestimate YSR at lower N/σ_v’. Therefore, another formula was proposed in Equation (2):

$$YSR = 19.029\left(\frac{N}{\sigma_v'(kPa)}\right)^{0.731} \tag{2}$$

This is represented by the red dashed line in Figure which was used to estimate YSR in this study.

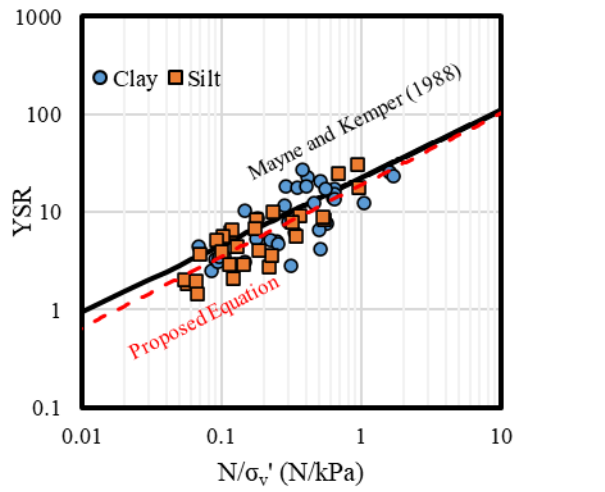


Figure 6. Correlation between SPT-N, effective stress, and YSR at Kediri (Pangaribuan, 2021)

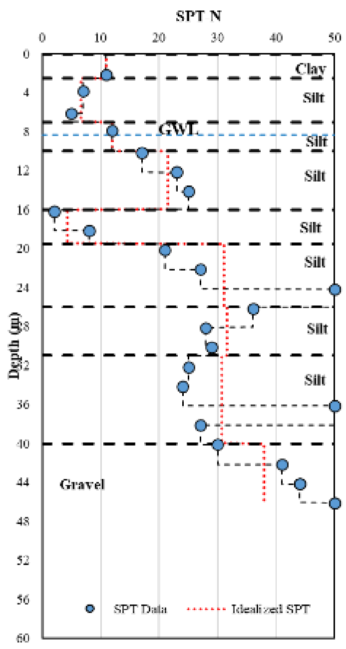


Figure 7. Soil Profile and N-SPT value

The in situ and laboratory tests conducted in the vicinity of the DDC pile and the static load tests on the piles were used to evaluate the performance of DDC. Meanwhile, the finite element analysis was applied through MIDAS GTS NX to back analyze the parameters of the materials.

The results of the soil stratification and SPT tests presented in Figure showed that most of the soils at the site are clay and silt from weathered tuff and they consist of residual soils.

The modified Mohr-Coulomb (MMC), commonly referred to as the hardening soil model, was used to model the in situ soil and DDC materials (Schanz *et al.*, 1999). Its notable features include three moduli which are the secant modulus with respect to 50% failure load (E_{50}), unloading-reloading modulus (E_{ur}), and constrained or oedometer modulus (E_{oed}). They are all non-linear concerning stress as indicated in the following equations:

$$E_{50} = E_{50}^{ref}\left(\frac{\sigma_1' K_0 + c\cot\phi}{p^{ref} + c\cot\phi}\right)^m \tag{3}$$

$$E_{ur} = E_{ur}^{ref}\left(\frac{\sigma_1' K_0 + c\cot\phi}{p^{ref} + c\cot\phi}\right)^m \tag{4}$$

$$E_{oed} = E_{oed}^{ref}\left(\frac{\sigma_1' + c\cot\phi}{p^{ref} + c\cot\phi}\right)^m \tag{5}$$

Where ϕ' is the effective friction angle, c is cohesion, E_{50}^{ref} is E_{50} at reference pressure (p^{ref}) which is commonly taken as 100 kPa, E_{ur}^{ref} is E_{ur} at p^{ref}, E_{oed}^{ref} is E_{oed}, m is power function which is taken as 1 for clay (Schanz *et al.*, 1999) and 0.5 for the gravel layer, σ_1' is the effective vertical stress, and K_0 is coefficient of at-rest pressure which is given as:

$$K_0 = OCR.K_0^{NC} - \frac{\upsilon_{ur}}{1-\upsilon_{ur}}\left(OCR-1\right) \tag{6}$$

$$K_0^{NC} = 1 - \sin\phi \tag{7}$$

Where υ_{ur} is Poisson's ratio for unloading-reloading and YSR was used as OCR. It is important to note that the modulus (E_{50}) in Table 1 is not under 100 kPa. Therefore, effective vertical stress (σ_v') at the site which is presented in Figure 8 and K_0 were used to calculate the E_{50}^{ref}.

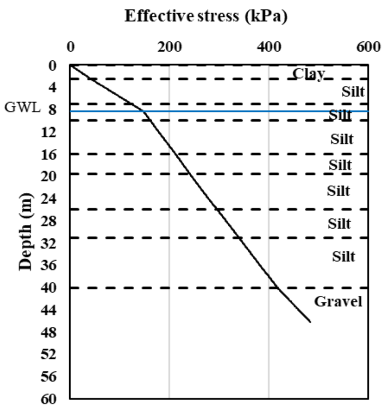


Figure 8. Effective stress versus depth

In the MMC model, the Poisson ratio unloading-reloading (υ_{ur}) can be set at 0.2 as recommended by Schanz *et al*., (1999). E_{ur}^{ref} is taken as $3E_{50}^{ref}$ while E_{oed}^{Ref} is equal to E_{50}^{ref} as typically used in practice. The correlation between SPT-N and Effective Young modulus (E') for coarse-grained soils was determined through the following equation (Schnaid, 2009):

$$\frac{E'}{N_{60}} = 1(MPa) \tag{8}$$

The correlation between SPT-N and undrained Young modulus (E_u) for fine-grained soils can be obtained through the following equation (Butler, 1975):

$$\frac{E_u}{N_{60}} = 1N(MPa) \tag{9}$$

Meanwhile, the correlation between undrained and drained modulus was evaluated using the following equation (Ameratunga *et al*., 2016):

$$E' = \frac{2}{3}(1+\upsilon)E_u \tag{10}$$

The SPT hammer at the site is automatic with approximately 75% energy and the SPT-N value in Figure 7 is required to be corrected to 60% energy (N_{60}) in order to estimate the E'. It was also assumed that E' equals E_{50}'. Moreover, the effective friction angle for fine-grained soils was obtained from Bjerrum and Simons (1960) as indicated in Figure 9. The correlation between plasticity index and friction angle from the triaxial CU at this site was also plotted in the same figure while the effective friction angle for coarse-grained soils was obtained from Peck, Hanson *et al*. (1974).

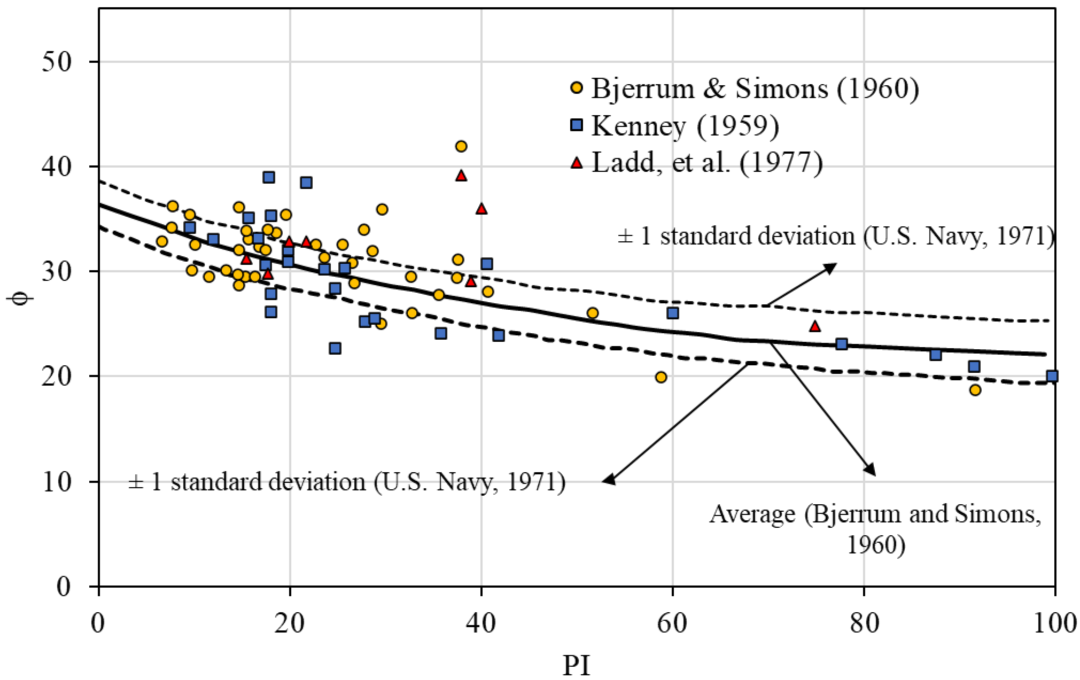


Figure 9. Correlation between Plasticity index, PI with friction angle (Bjerrum and Simons, 1960; Kenney, 1959; Ladd *et al*., 1977)

The correlation from Sorensen and Okkels (2013) was adopted to obtain effective cohesion (c'). Meanwhile, Sorensen and Okkels (2013) proposed that c' can be conservatively estimated for over-consolidated soils using the following equation:

$$c' = 0.1s_u \tag{11}$$

The undrained shear strength of the in situ soil was determined through the application of a pressure meter test at the site based on the correlation proposed by Amar and Jézéquel (1972). The undrained shear strength was later correlated with the SPT-N value as indicated in Figure 10.

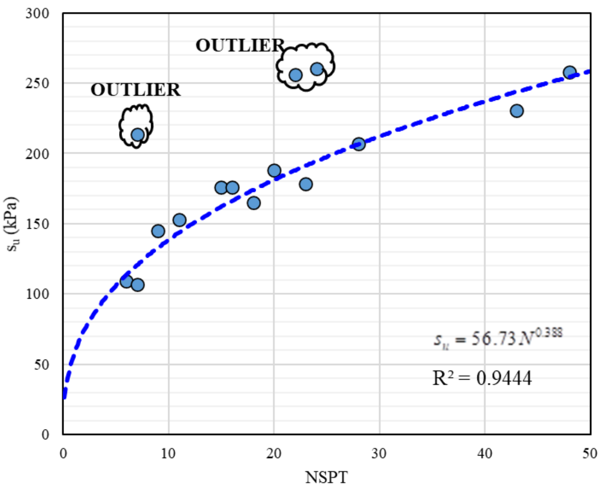


Figure 10. Correlation between undrained shear strength and SPT-N for the soil at the site

Table 1 shows the adopted soil properties and the SPT-N value adopted for the soil surrounding the DDC pile where γ is the unit weight of soil interpreted from the value typically used (Budhu, 2010), LL is the liquid limit, and PL is the plastic limit. Meanwhile, Table 2 shows the interpreted parameters for the MMC model.

The finite element (FE) model was applied to the trial test on DDC using MIDAS GTS NX and the results are presented in Figure 11. The boundary was extended up to 7D in order to avoid the boundary effect and simulate a single DDC pile. Moreover, an interface was also applied between the DDC pile and the surrounding soil to allow separation between soil and DCC nodes. It is important to note that the pile was loaded up to 900 kN and modeled using uniformly distributed stress at the top.

Table 1. Soil properties

Sample	Depth	γ	N	N_{60}	LL	PL	PI
	m	kN/m³					
Clay	0	16.7	11	14	53	26	28
Silt	2.5	18.2	7	8	40	28	12
Silt	7	18.2	12	15	41	28	13
Silt	10	18.4	21	27	41	28	13
Silt	16	18.1	4	5	41	28	13
Silt	19.5	18.6	31	39	41	28	13
Silt	26	18.6	32	39	58	36	22
Silt	31	18.6	31	38	41	28	13
Gravel	40	20.8	38	47	-	-	-

Table 2. Interpreted parameters for the MMC model

Sample	Depth	σ_v'ave	ϕ	c'	C_u	YSR	E_u	E_{50}	K_0	σ_3	$E_{50,Ref}$	$E_{Oed,Ref}$	$E_{UR,Ref}$	m
	m	kPa	⁰	kPa	kPa		kPa	kPa		kPa	kPa	kPa	kPa	
Clay	0	21	29	14.4	143.7	12.0	14000	14000	3.4	71	18147	14000	54442	1
Silt	2.5	83	33	11.8	118.1	3.0	8000	8000	0.9	73	10355	8000	31066	1
Silt	7	146	32	14.9	148.6	3.1	15000	15000	0.9	133	11820	15000	35461	1
Silt	10	187	32	18.6	186.2	3.9	27000	27000	1.1	205	14934	27000	44801	1
Silt	16	226	32	10.0	99.8	1.0	5000	5000	0.5	108	4683	5000	14049	1
Silt	19.5	268	32	21.5	215.0	3.9	39000	39000	1.1	295	15867	39000	47600	1
Silt	26	318	30	21.6	216.3	3.5	39000	39000	1.1	358	13559	39000	40677	1
Silt	31	378	32	21.4	213.9	3.0	38000	38000	0.9	343	13507	38000	40520	1
Gravel	40	449	38			1.0		47462	0.4	172	36188	47462	108563	0.5

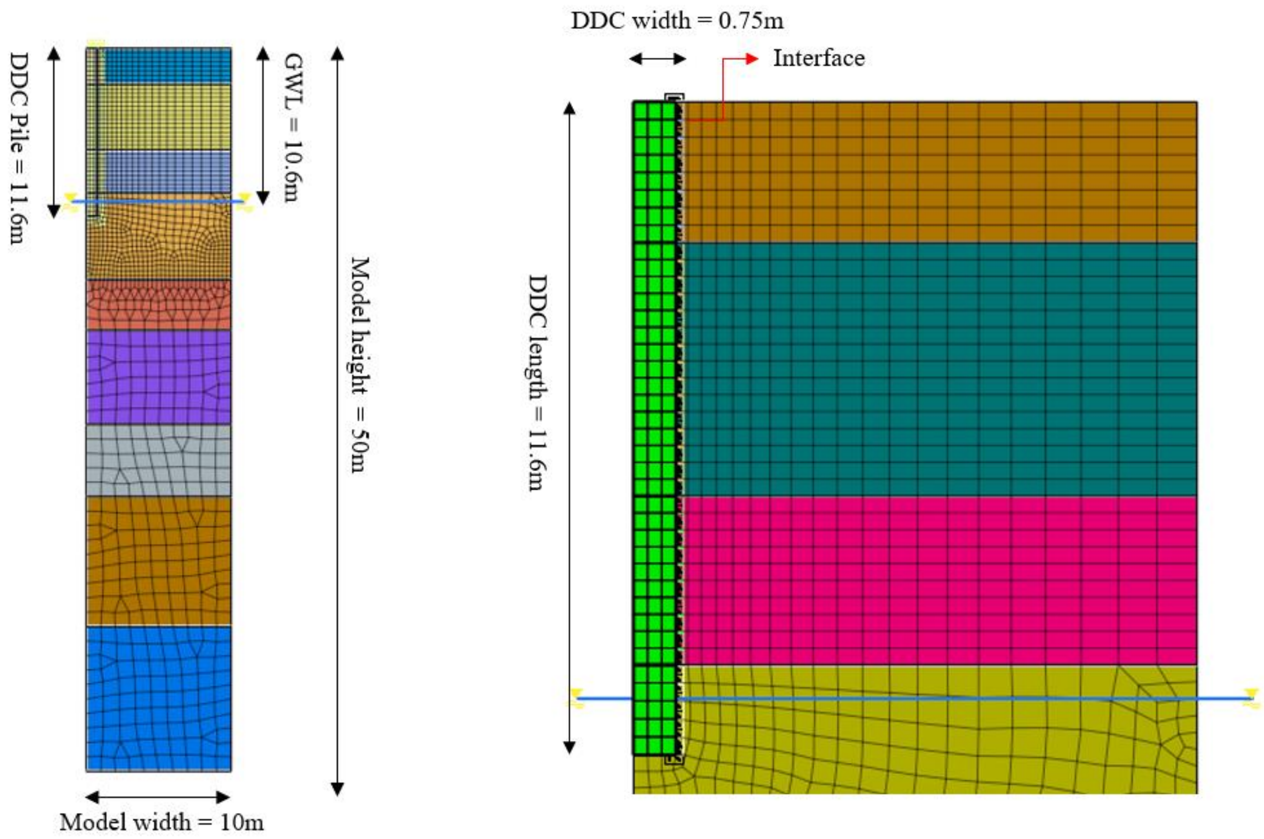
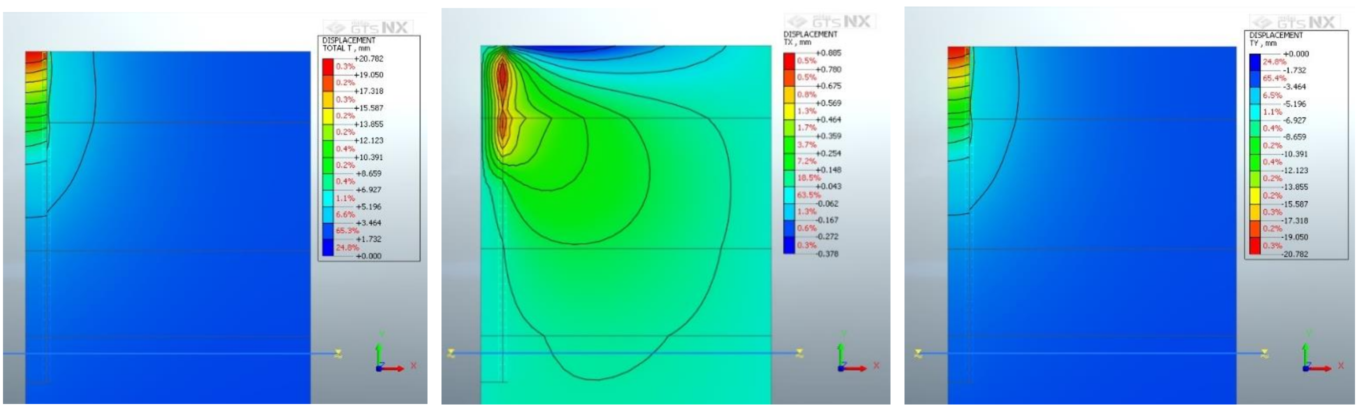


Figure 11. FE Model for DDC



(a) Total displacement (b) Lateral displacement (c) Vertikal displacement

Figure 12. DDC pile displacement (a). Total displacement, (b). Lateral displacement, and (c). Vertical displacement

3 RESULTS

Figure 12a shows the total displacement of the DDC pile under 900 kN axial load while the lateral and vertical displacement is indicated in Figure 12b and c, respectively. Moreover, Figure 13 shows the result for the back analysis, and the properties of the pile were adjusted up to the period they match with the S4-TP401. It is important to note that the materials were obtained from the rock excavation to obtain a conservative estimate. The back analysis results showed that E_{50} is 50 MPa, E_{Oed} is 50 MPa, E_{UR} is 150 MPa, c' is 5 kPa, and ϕ is 43^0.

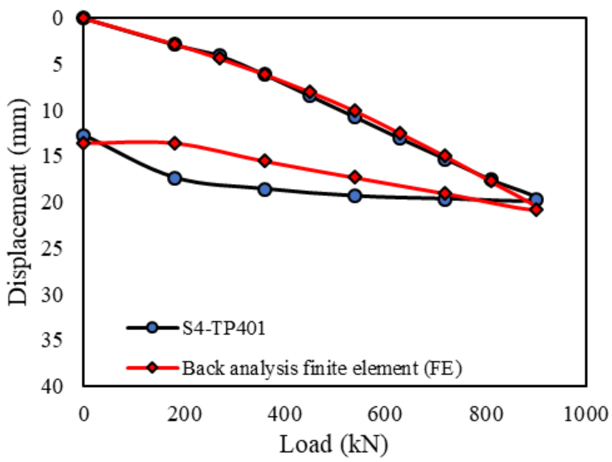


Figure 13. Back analysis result using FEM on DDC piles

4 DISCUSSION

Figure 12 shows that DDC piles move in both vertical and lateral directions with the lateral displacement observed to have occurred at the edge of the piles which represents the bulging mechanism as indicated in Figure 12b. Furthermore, significant settlement was observed to have occurred at the top of the pile and this slightly affected the surrounding soil as presented in Figure 12c. It was also discovered in Figure 13 that the back analysis was able to represent the loading mechanism with a maximum accuracy of 4 mm recorded at the unloading range when the soil was unloaded to 180 kN axial forces. However, the DDC pile is expected to be subjected to only axial loads during construction and this makes the unloading part insignificant to the analysis.

5 CONCLUSION

Downhole dynamic compaction (DDC) has been employed as one of the ground improvement methods in Indonesia and proved to be useful specifically when the on-site materials are oversized. It is challenging to determine DDC parameters to be used in a design. Therefore, this study conducted back analysis based on the finite element method to determine these parameters, and the modified Mohr-Coulomb (MMC) was observed to be suitable to model the DDC. The load-deformation curve from the back analysis also matched the field measurement effectively.

DISCLAIMER

The authors have no conflict of interest during the process of publishing this study.

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