

Behavior of Single Pile and Group Piles Under Negative Skin Friction Based on Field Strain Measurement and Assessment using 3D Finite Element Analysis

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ABSTRACT: When pile foundation is embedded through consolidating soils, the settlement of soils above neutral point will drag the pile shaft down and produces negative skin friction (NSF) or down-drag force that reduces the allowable axial capacity. The magnitude of NSF is influenced by pile dimension, soil shear strength, and the depth of consolidating layer. The Authors investigated this downdrag using CPTu based on sleeve friction and pore pressure ratio (B_q or B_q^* values) to determine the degree of consolidation. This paper is intended to discuss pile behaviour under negative skin friction for single pile and group pile by case studies. The first case study is single pile in reclaimed area in Semarang (Central Java), where a single spun pile diameter 600mm and 42m depth was instrumented using fibre optic (FO) along the pile to measure mobilized NSF during consolidation until the ultimate NSF has been reached. The result show unexpectedly very high NSF reaching about 1850 kN. Soil-structure interaction of pile groups is analyzed and discussed in this paper. From this study, the mechanism of NSF for group piles is investigated, where numerical analysis of a large group piles at the same location is conducted using the 3D Finite Element Method. The down-drag force for the pile in the middle of the pile group is smaller compared to the pile at the edge, where there is a constraint effect between the piles to the soil settlement.

KEYWORDS: Negative Skin Friction, Downdrag Force, Spun Pile, Underconsolidating Soils, Fibre Optic Instrument

1 INTRODUCTION

Settlement occurring in the surrounding soil layer can influence the performance and bearing capacity of an embedded pile foundation. The settlement of the soil surrounding the pile may occur due to several phenomena, including soil consolidation induced by surcharge loads, geological processes where soil compression induced by sedimentation, settlement induced by the lowering of the groundwater table in soft clays, or ground settlement triggered by liquefaction events (Pusat Studi Geoteknik, 2025).

If the settlement of the surrounding soil is greater than the settlement of the pile, the soil layer undergoing larger settlement will drag the pile shaft downward. This results in the development of downward shear stress along the pile, known as negative skin friction (NSF) or downdrag force. The segment of the pile shaft that is subjected to negative skin friction (NSF) is located about the neutral point. The neutral point is a position or elevation of a pile where the settlement of the pile equals the settlement of the surrounding soil, as illustrated in Figure 1.

Negative skin friction (NSF) of downdrag force reduces the bearing capacity of a pile, where the soil resistance above the neutral point is lost and the negative friction acts as additional downward force. When NSF is significant, it may result in negative bearing capacity, where the pile foundation no longer supports the structural load but act as a downward force on the superstructure.

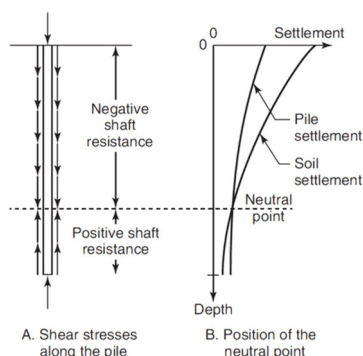


Figure 1. Negative skin friction on pile foundation (Briaud, 2013)

The magnitude of the downdrag force can be determined by multiplying the unit negative shaft resistance by the surface area of the pile segment located above the neutral point. The resulting downdrag force will differ between friction piles and end-bearing piles. This difference arises because end-bearing piles typically experience less settlement than friction piles. The smaller the settlement of the pile, the deeper the position of the neutral point. A deeper neutral point results in a longer segment of the pile being subjected to NSF, thereby increasing the magnitude of the downdrag force. Hence, end-bearing piles are likely to experience greater NSF forces than friction piles. The magnitude of the downdrag force acting on a pile depends on several factors, including pile characteristics (pile type, installation method, length, cross-sectional shape), soil characteristics (shear strength, compressibility, and depth of soft soil layer), the cause of ground movement, the degree of consolidation during pile installation, the magnitude and duration of surcharge loading.

Kuwabara and Poulos (1989) stated that the amount of ground settlement required to mobilize NSF varies from 0.5% of the pile diameter for short piles to up to 5% of the diameter for long piles. Briaud and Tucker (1997) proposed several conditions in which the negative skin friction phenomena can occur.

- The total settlement of the ground surface will be larger than 100 mm.
- The settlement of the ground surface after the piles are installed will be larger than 10 mm.
- The height of the embankment to be placed on the ground surface exceeds 2 m
- The thickness of the soft layer is larger than 10 m
- The water table will be drawn down by more than 4 m
- The piles are longer than 25 m.

In the practical applications in Indonesia, the relative displacement resulting from the consolidation process of the compressible soil layer is the dominant factor influencing the occurrence of the negative skin friction (Alvi and Rahardjo, 2021).

2 CASE STUDY OF NEGATIVE SKIN FRICTION ON SPUN PILE IN RECLAIMED AREA

2.1 Case study on reclaimed area in Semarang, Indonesia

This study is based on a reclaimed area located in the northern part of Semarang City, Indonesia, where building and infrastructure construction about 15 years after reclamation. The area is planned to develop into a central business district in Semarang. The primary geotechnical challenge in this reclaimed area is the existence of a thick layer of very soft marine clay beneath bouldery clay as the fill material. As a result, the consolidation process in the very soft marine clay layer was still ongoing, with a degree of consolidation of less than 70% at the time of foundation construction. This paper presents a case study of a high-rise building in this reclaimed area, where spun pile group system with a diameter of 600 mm was employed as the foundation system. The installation of spun piles in an underconsolidation soil layer induces the development of negative skin friction (NSF) along the piles.



Figure 2. Location of reclaimed area

2.2 Soil condition

Geotechnical investigation was conducted in 2023 to characterize the subsurface soil layers and their properties. The in-situ testing program include borehole drilling with Standard Penetration (SPT) and Cone Penetration Test with pore pressure measurement (CPTu). The CPTu provide data on excess pore pressure, which is used to interpret the degree of consolidation. This information confirms whether the consolidation process induced by the fill material is still ongoing or has been completed. Rahardjo et. al. (2024) established the CPTu is useful to interpret underconsolidating soils for various reclamation cases in Indonesia. The results from both SPT and CPTu data (Figure 3 and Figure 4) are consistent and indicate that the upper layer consists of gravelly sand as fill material with an approximately thickness of 13 m. Beneath the fill layer is very soft marine silty clay with, approximately 12 m thick. The soil layer below the soft soil is medium to stiff silty clay with very dense sand lens.

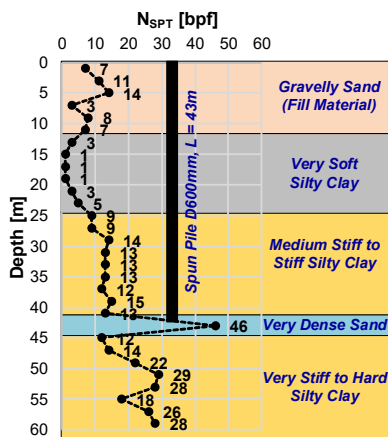


Figure 3. Soil profiles from boring and SPT data (PT GEC, 2023)

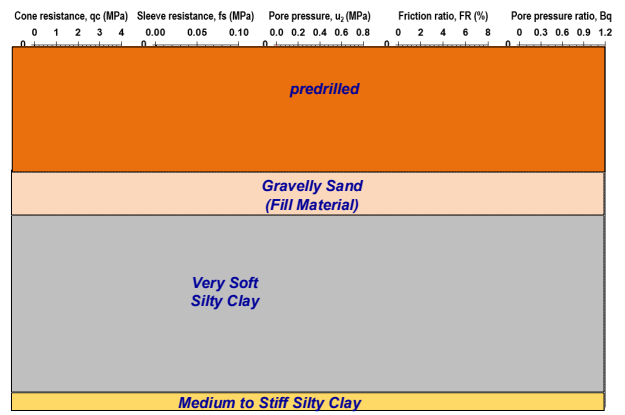


Figure 4. Soil profiles from CPTu data (PT GEC, 2023)

2.3 Assessment the degree of consolidation based on CPTu

The Cone Penetration Test with pore pressure measurement (CPTu), also known as the Piezocone Test, records three key parameters: cone resistance (q_c), sleeve friction (f_s), and pore pressure (u_2). In this study, CPTu utilized to assess the degree of consolidation within the soft soil layer. The measured pore pressure (u_2), by pore pressure filter located behind the cone tip, consists of three components: hydrostatic pore pressure (u_0), excess pore pressure generated by cone penetration, and excess pore pressure associated with the in-situ soil condition.

Rahardjo et. al. (2008) proposed a method for estimating excess pore pressure in the soil using dissipation curve, which can be used to evaluate the degree of consolidation. Subsequently, Rahardjo et. al. (2014) developed a method to estimate the degree of consolidation by correlating it with the pore pressure ratio parameter (B_q). The pore pressure ratio (B_q) is defined as the ratio of the measured excess pore water pressure ($u_2 - u_0$) to the net resistance ($q_t - \sigma_{v0}$), where σ_{v0} is the initial vertical total stress.

$$B_q = \frac{u_2 - u_0}{q_t - \sigma_{v0}} \quad (1)$$

The correlation between the degree of consolidation (%U) to the pore pressure ratio (B_q) is shown in Figure 5. This correlation shows that the soil is underconsolidating when the B_q is more than 0.75.

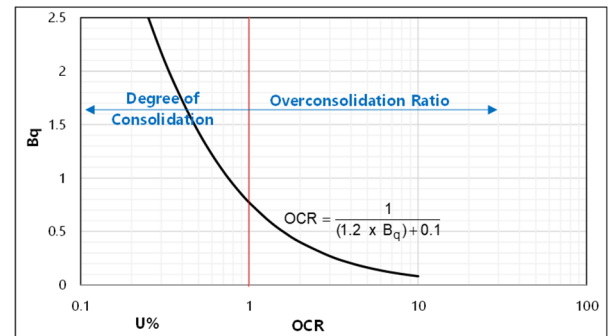


Figure 5. Relationship between degree of consolidation (or OCR) and pore pressure ratio based on CPTu measurement (Rahardjo et. al., 2014)

The use of pore pressure ratio parameter (B_q) has limitation, as it is highly dependent on the assumption of input soil unit weight. To address this issue, Rahardjo et. al. (2016) proposed a new parameter, the modified pore pressure ratio (B_q^*), as an alternative to the conventional pore pressure ratio (B_q). The modified pore pressure ratio (B_q^*) is defined as the ratio between the measured pore pressure response (u_2) and total cone resistance (q_t), and it is derived from measured values independent of assumption regarding soil unit weight. This

parameter directly represents the proportion between pore water pressure and total resistance, offering a more objective and reliable assessment of soil behavior.

$$B_q^* = \frac{u_2}{q_t} \quad (2)$$

Following the study by Rahardjo et al. (2016), Rahardjo and Setiawan (2017) improved the correlation database by incorporating data from the Sidoarjo Mud. Relationship between the degree of consolidation (%U) or OCR and modified pore pressure ratio (B_q^*) is shown in Figure 6.

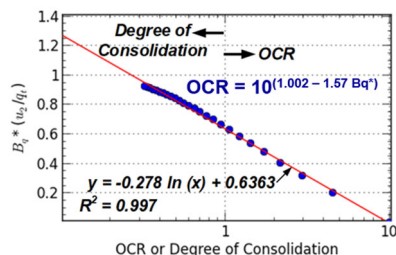


Figure 6. Relationship between degree of consolidation (or OCR) and modified pore pressure ratio (Rahardjo et. al., 2016; Rahardjo and Setiawan, 2017)

Equation (3) and (4) present the formulas used to calculate the degree of consolidation based on the B_q method (Rahardjo et. al., 2014) and the modified B_q^* method (Rahardjo et. al., 2016), respectively.

$$OCR = \%U = \frac{1}{1.2 B_q + 0.1} \quad (3)$$

$$OCR = \%U = 10^{(1.002 - 1.57 B_q^*)} \quad (4)$$

Based on the CPTu data obtained at the site, the average degree of consolidation for the very soft soil layer is estimated to be 79 % using B_q method and 76% using the modified B_q^* method. These values indicate that the soft soil remains consolidating, and as a result, negative skin friction is expected to develop along the pile.

3 MEASUREMENT OF NEGATIVE SKIN FRICTION OF SINGLE PILE USING FIBRE OPTIC INSTRUMENT

3.1 Scheme of Instrumented Pile Test and Measurement

This study was conducted on a single 600 mm diameter spun pile with a length of 43 meters, in which a fiber optic instrumentation cable was installed through the central bore of the pile. The type of fiber optic system used was Brillouin Optical Time Domain Analysis (BOTDA), which measures the frequency shift of a laser signal (Pelecanos et. al., 2018). The recorded frequency changes are converted into material strain values using a predetermined calibration factor. As a result, axial strain along the pile can be accurately measured using this instrumentation (Mohamad et. al., 2019).

Following the installation of the fiber optic cable, an axial load test was performed by applying a load equal to 300% of the design load ($300\% \times 1250 \text{ kN} = 3750 \text{ kN}$) at the pile head. During the load test, strain measurements were taken at each loading stage. The strain data obtained during this controlled load test represent the behavior of the pile under load without the influence of negative skin friction (NSF). The strain behavior was then interpreted into load transfer curves for each load increment applied to the pile head.

After the load testing phase, strain monitoring was carried out over several months to capture the strain response of the pile as soil consolidation occurred surrounding it. This allowed for

the acquisition of strain data under NSF conditions. The measured strain under zero head load conditions was further analyzed to estimate the magnitude of NSF acting on the pile.

One of the main challenges in fiber optic instrumentation of a spun pile lies in the pile's factory-fabricated and segmental construction. To ensure that the measured strain accurately reflects the actual structural strain of the pile, the fiber optic cable was housed within a steel pipe inserted into the central hole of the pile. During installation, the fiber optic cable was secured using cable ties fixed to elbow brackets on the four inner sides of the steel pipe. Each steel pipe segment was welded at the joints to maintain continuity. Grout was used to fill the annular space between the steel pipe and the pile, serving to provide adequate bonding and strain transfer between the pipe and the pile structure.



Figure 7. Cable attachment during steel pipe insertion into hollow part of the spun pile



Figure 8. Grout with high strength concrete between steel pipe in the middle and *spun pile*

3.2 Instrumented axial pile load test results

The installation of the spun piles was carried out using a Hydraulic Static Pile Driver (HSPD), in which the 43m of pile length were pressed into the ground by hydraulic pressure. Two weeks after pile installation, a static axial pile load test was conducted by applying a load of 300% of the design load ($300\% \times 1250 \text{ kN} = 3750 \text{ kN}$) at the pile head. The test conducted using cyclic loading procedure, involving loading and unloading at every 50% increment of the applied load, resulting in six load-unload curves corresponding to each stage of loading. The results of the pile load test are presented in the load-settlement curve shown in Figure 9. The total duration of the test was 38 hours. Within this relatively short period, there is no negative skin friction (NSF) occurred. Therefore, the test results can be considered free from NSF influence, minimizing the risk of misinterpretation due to time-dependent drag forces.

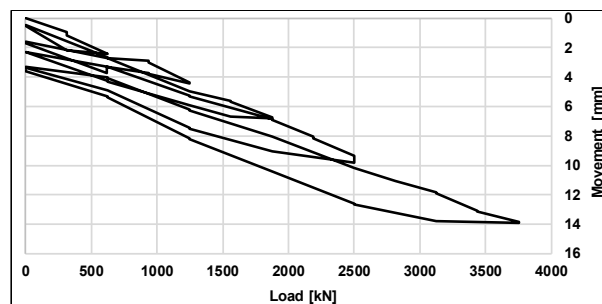


Figure 9. Measured load-settlement curve during axial load test

The strain data measured during the loading test are presented in Figure 10 (left). These data provide several insights, including:

- The applied load of 3750 kN does not induce strain in the lowest segment of the pile. This indicates that the maximum applied load was not sufficient to fully mobilized the skin friction resistance along the bottom segment of the pile.
- An excessive strain was observed at a depth of 14 m. There are two (2) possible explanations for this anomaly: (1) a reduction of grouting quality during field installation, or (2) inadequate quality at the pile joint, as this depth corresponds to the segment connection. Despite this irregularity, interpretation of the overall pile behavior remains feasible.
- The maximum measured strain was 580 micro-strain ($\mu\epsilon$), which remains well below the structural safety threshold. The upper limit for compressive strain in structural concrete generally accepted to be 3000 $\mu\epsilon$.

The load transfer curve shows the distribution of axial load along the pile for each applied load level. As the load applied at the pile head increases, the load is transferred progressively deeper along the pile, resulting in greater mobilization of soil resistance around the pile. The load transfer curve along the pile is presented in Figure 10 (right).

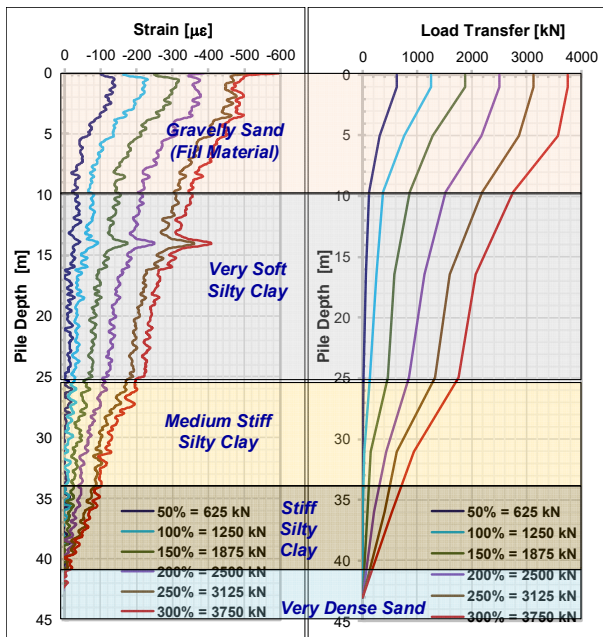


Figure 10. Measured axial strain data from Fiber Optic (left) and load transfer (right) during pile load test

3.3 Strain measurement results of pile subjected to negative skin friction (NSF)

After the pile load test was completed, strain in the spun pile was continuously monitored using fiber optic instrumentation over a period of several months to assess potential changes in strain resulting from the ongoing soil consolidation process. If changes in strain are observed over time in the absence of external loading, such changes are attributed to downdrag forces acting on the pile during consolidation. Fiber optic monitoring was conducted six (6) times following the pile installation on March 11, 2023, specifically on June 1, 2023; June 16, 2023; June 28, 2023; August 19, 2023; September 13, 2023; and October 5, 2023.

The negative skin friction (NSF) measured using fiber optic instrumentation is represented by the load transfer curve

along the pile, as shown in Figure 11. The NSF is quantified based on the portion of the load transfer curve from the pile head down to the neutral point. The neutral point is defined as the depth at which the settlement of the surrounding soil equals the settlement of the pile. In Figure 11, the neutral point is identified as the inflection point of the curve—above this elevation, the transferred load decreases. The neutral point is located at a depth of 34 meters. The downdrag force is estimated to be approximately 1170 kN on June 1, and increased to 1850 kN by October 5, as shown in Figure 12. Therefore, the maximum measured downdrag force on this pile is 1850 kN. The upper 10 meters from the pile head indicate a downdrag force of approximately 1100 kN due to the granular fill material, contributing for about 60% of the total downdrag force (1850 kN).

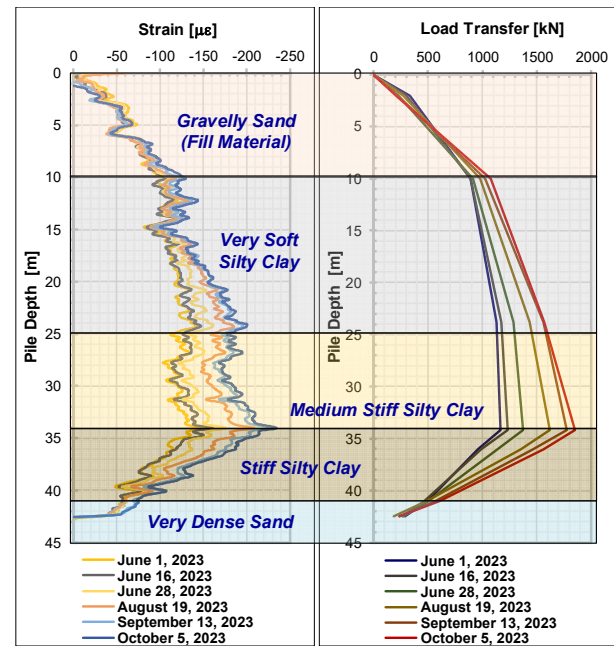


Figure 11. Load transfer curves measured by fibre optic instrumentation during the consolidation process

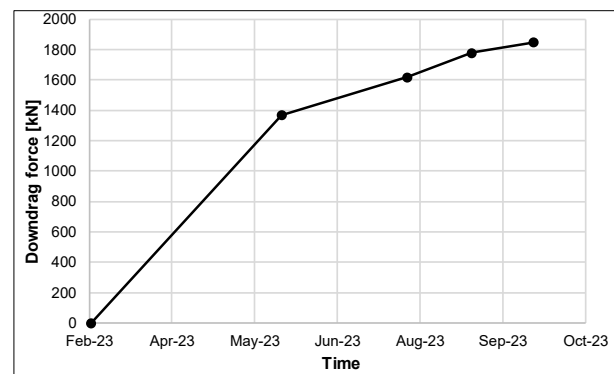


Figure 12. Mobilized downdrag force over-time after pile installation

3.4 Comparison of τ - z curve during pile load test (without NSF) and during consolidation process (with NSF)

The actual τ - z curve is obtained based on the response of changes in mobilized skin friction to the displacement that occurs for each pile segment from FO strain measurement. The τ - z curve also represents the mobilized skin friction (τ) for each depth transition to pile displacement. Although the negative skin friction (NSF) is associated with soil settlement, the main focus of the τ - z curve is the relative displacement between the pile and the surrounding soil.

In soft soil layers, the mobilized positive skin friction during the pile load test differs from the mobilized negative skin friction (NSF) observed during the consolidation process, as shows in Figure 13. The test results reveal an important finding: the maximum mobilized negative skin friction is slightly lower than the maximum mobilized positive skin friction induced by the applied working load. This difference arises due to the distinct loading mechanisms in each case. During the pile load test, the load is applied at the pile head, whereas during the consolidation process, the downdrag force develops along the pile segment above the neutral point. An important finding is that the mobilization of negative skin friction (NSF) is highly dependent on the relative displacement between the pile and the surrounding soil. A relative displacement in the range of approximately 3 to 8 mm is generally sufficient to fully mobilize the ultimate NSF.

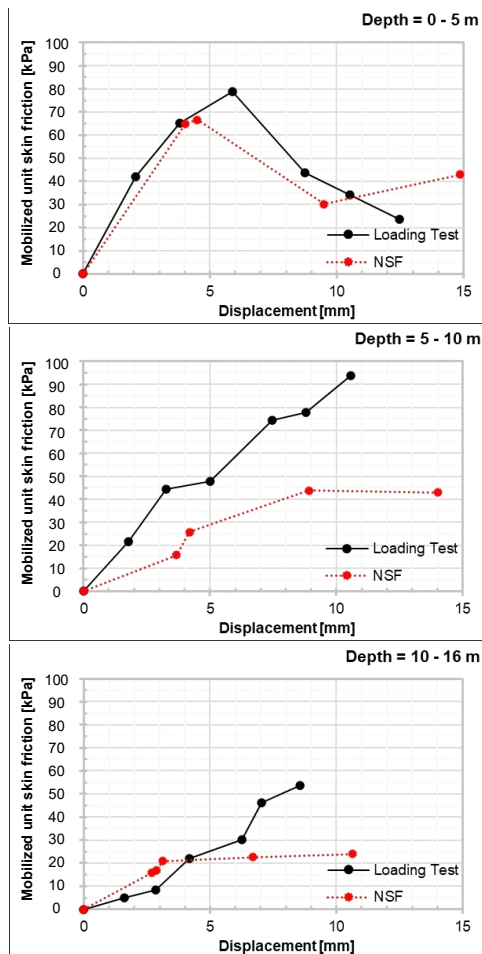


Figure 13. Comparison between typical τ -z curve during pile load test (positive skin friction) and during consolidation process (negative skin friction / NSF)

4 STUDY ON THE BEHAVIOR OF A PILE GROUP UNDER NEGATIVE SKIN FRICTION USING 3D FINITE ELEMENT METHOD

4.1 Analysis Method using 3D Finite Element Method

A study on the pile group under negative skin friction is conducted at the same case with the study of the single pile in the previous discussion. To evaluate the impact of negative skin friction on single pile versus pile group, a single pile and a pile group consisting of 207 piles were modelled within the same 3D Model.

The analysis is conducted using the 3D Finite Element Method (FEM) implemented in the MIDAS GTS NX Program. Soil materials were modeled in cluster, with the Soft Soil Model used for clayey layers and the Hardening Soil Model for sandy layers. The pile cap was represented as an elastic solid cluster using concrete material properties, while the piles were modeled as embedded beam elements. The thickness for each soil layer was defined according based on the drilling data. The study was performed in four (4) stages:

- Stage 1: Calibration of soil parameters to obtain similar downdrag force magnitude based on strain measurement.
- Stage 2: Determination of the final settlement induced by the existing embankment.
- Stage 3: Modeling of pile installation when the degree of consolidation of 76% (from the CPTu test). For the simulation, the degree of consolidation was expressed as the ratio of the estimated settlement that had occurred to the total expected final settlement.
- Stage 4: Evaluation of negative skin friction, initiated from the time of pile installation at 76% consolidation. The analysis focused on the development of NSF along the pile above the neutral point during consolidation process.

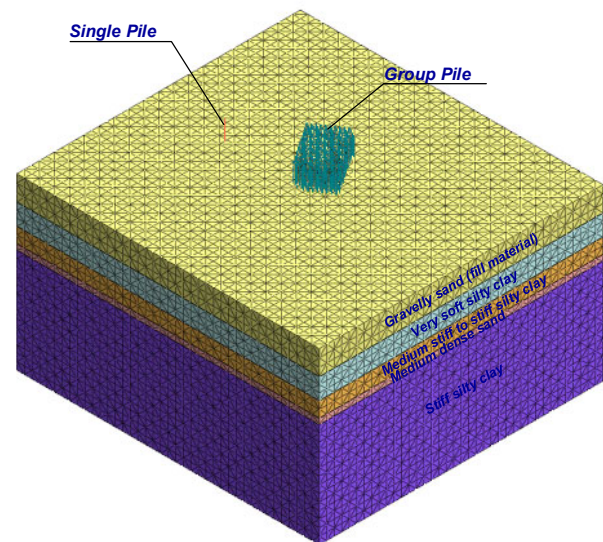


Figure 14. Soil stratification and pile modeling using 3D FEM

4.2 Analysis Results and Discussion

The results of the analysis are presented in the form of settlement contours due to the consolidation process in the soft clay layer (Figure 15), as well as pile settlements resulting from downdrag forces.

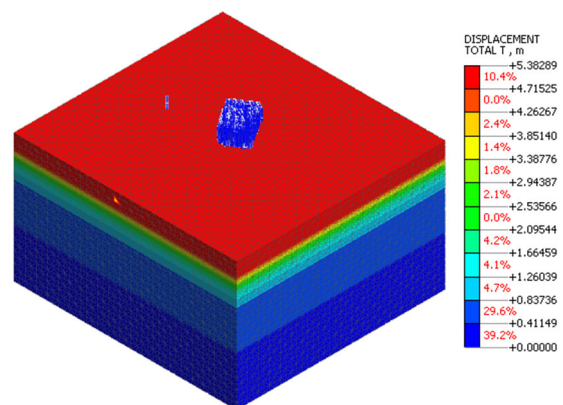


Figure 15. Soil settlement contour from 3D FEM

In this modeling, there is a difference in magnitude between the field measurement (1850 kN) and the back-analysis result (1880 kN) for the single pile. This discrepancy, which is less than 10%, is attributed to the constitutive model and parameter assumptions used in the analysis. Based on the load transfer analysis for both single piles and pile groups (see Figure 16), it was observed that the magnitude of negative skin friction (NSF) on piles within group is smaller than single pile. In addition, piles located in the inner side (interior) of the group exhibit lower NSF compared to those on the perimeter. This finding supports the conclusions drawn by Briaud and Tucker (1997) and Basile (2018), which indicate that the downdrag forces are reduced in a pile group configuration.

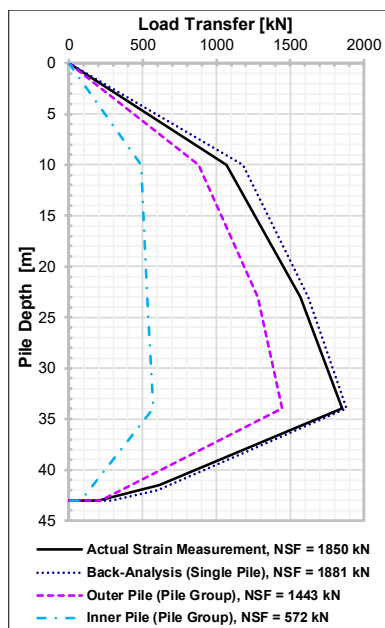


Figure 16. Load transfer for single pile and pile group

5 CONCLUSIONS

Consolidating soils present a significant challenge for pile foundations, as the resulting downdrag force or negative skin friction (NSF) that reduce the axial capacity of the pile. Field strain measurements provide important information of the actual magnitude of downdrag force and the location of the neutral point. These measurements also confirm that the behavior of piles under downdrag is primarily determined by the mobilized skin friction, which is a function of the relative displacement between the pile and the consolidating soil. A relative displacement in the range of approximately 3 to 8 mm is generally sufficient to fully mobilize the ultimate NSF.

The magnitude of NSF along piles within a group is lower than that observed in single pile conditions. Moreover, piles located in the interior of a pile group experience lower downdrag forces compared to those positioned at the outer edges.

6 ACKNOWLEDGEMENTS

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