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Field Monitoring of Secant-Pile Supported 11 m Excavation in Residual Soil: A Case Study from Bogor

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ABSTRACT To maximize usable construction area in contoured terrain, cut-and-fill operations are often unavoidable. This paper presents a two-tier excavation, the deepest cut being 11 m + 4.7 m and the shorter cut being 5 m + 4 m. The cut was used to make space for a residential house. Subsurface investigation revealed a stratigraphy of medium silt (average NSPT ≈ 5) to a depth of 6 m, underlain by 4 m of stiff silt (average NSPT ≈ 12), and a hard silt layer extending beyond the investigation depth. The deepest excavation till mid-excavation was retained by 22 m long secant pile, while the mid-excavation till the shallowest excavation was retained by 16 m long secant pile. Excavation support was provided by secant piles: a 22 m long system for the upper excavation and a 16 m long system for the lower excavation, comprising 0.8 m diameter secondary piles and 0.6 m diameter primary piles. Pre-construction analysis predicted a maximum deformation of 2.3 cm at the top of the secant pile for the deepest cut. To ensure deformation was within limits, inclinometer monitoring was conducted daily over 12 days during excavation. Surprisingly, no measurable movement was recorded. This could be attributed to the saturated soil parameters used for analysis, whereas the soil was in unsaturated state during excavation. While adopting unsaturated parameters may yield results closer to observed behavior, such an approach is less conservative. Given Bogor's high rainfall, conservative design using saturated soil parameters remains prudent to ensure safety, particularly in residential construction.

KEYWORDS Excavation; Secant Pile; Residual Soil; Inclinometer Monitoring; Field Monitoring

1 INTRODUCTION

1.1 Need for Excavation

Excavation projects are usually done in urban area for construction of building basements, subway stations, and so on (Ou, 2006). Excavation on sloping or mountainous terrain is required for road formation, building platforms, and hillside development (Terzaghi et al., 1996). Any face cut into colluvium, residual soil or weathered rock in such terrain is a soil cut slope, and its stability governs both the safety works and their long-term maintenance cost (Caltrans, 2026). Where the works are executed as an open cut, there are two typical options: batter slope or retain it vertically.

Batter slope is normally the cheaper option, where it is available. It requires no retaining structure, no support system, and minimal temporary design work. However, the cost and working space does not scale linearly with depth. The horizontal working space required is at least the excavation height multiply by $\cot \beta$, in which β typically range from 26.6° to 33.7° (1V:2H to 1V:1.5H). In addition, the excavated volumes grow approximately with square of the depth (Terzaghi and Peck, 1967). Meaning, for deeper cuts, the space required can be too wasteful. For slopes steeper than 1V:1H, it is necessary to perform stability analyses and retaining structure is usually needed.

A retained vertical cut becomes necessary where the natural ground slope is steep, where the site boundary limits; the volume of temporary cut and backfill is unacceptable on cost; or environmental regulations; the cut face would be exposed while in wet season particularly in the humid tropics and subtropics, where deep weathering produces residual soil profiles and rainfall infiltration is the dominant landslide trigger (Rahardjo et al. 2007). Hillslope excavation does not only expose the slope to rainfall, but also makes the problem worse. Field monitoring and back-analysis of cut slope failures show two effects acting together. Rainfall infiltrates the slope and reduces soil suction, which lowers the shear strength. The steep cut face then allows larger displacements to develop in the hillslope. Together they trigger landslide (Pradhan et al., 2022).

1.2 Brief Project Description and Choice of Retaining System

In this case study, the excavation was done in Bogor, Indonesia, which predominantly consist of volcanic residual soil. The slope was cut to make way for a residential house. As recommended by Rahardjo et al. (2007), for cases that exposes cut face to wet season, retaining system is needed. There are many factors affecting choice of retaining system, such as retained height, ground and groundwater conditions, the stability of the wider slope, constructability on a sloping site, and cost (Ou, 2006; Gaba et al., 2017; Caltrans, 2020). Two features of hillside cut narrow the choices immediately relative to a pit excavation.

Firstly, a hillside or slope excavation is one-sided. Meaning, there is no opposing face to strut against, so internal bracing is unavailable and lateral support must come from the cantilever action, ground anchors, or soil nails (Lazarte et al., 2015; Gaba et al. 2017). Secondly, the in-situ ground conditions consist of residual soil, colluvium or weathered rock rather than engineered fill, and the ground surface behind the wall may not be necessarily flat, i.e., sloping upward. Both conditions raise the active earth pressure relative with level ground. The wall is only one element inside the slope, and the stability of that slope must be checked on its own.

Soil nail and ground anchor reinforce the ground in place rather than retaining it, and constructed top-down in lifts as the soil is cut (Lazarte et al. 2015; Caltrans, 2021). Soil nail and ground anchor are commonly used in hillside excavation, however, if there are adjacent property behind the excavation, the use of ground anchors is restricted.

Another retaining system commonly used for hillside excavation is contiguous bored pile wall (also called soldier pile wall), which consists of reinforced concrete piles installed in row with small gaps between them, usually anchored, and are the system most often used where the piles must contribute to global stability (Reese et al., 1992; Poulos, 1995). When watertight wall is required, secant pile walls can be used. In secant pile walls, primary piles (which is unreinforced) overlap with the secondary piles (reinforced piles) to form continuous barrier, providing watertight wall, but at additional cost (Gaba et al., 2017). At modest heights, gabion or reinforced concrete cantilever walls can remain competitive. Diaphragm walls are rarely justified in this setting because of mobilization cost and access constraints on hillside sites (Ou, 2006; Puller, 2003).

For the present case, secant pile was adopted for its water tightness and preventing fines to be washed off between piles.

2 PROJECT DETAILS AND NUMERICAL ANALYSIS

2.1 Detail of Excavation and Soil Profile

Figure 1 shows the plan view of the site to be excavated. A total of 4 boreholes along the location of secant piles were conducted. The secant pile wall consisted of 0.6 m diameter primary pile (pile without steel reinforcement), and 0.8 m diameter secondary pile (pile with steel reinforcement), spaced at 1.2 m interval. There were two configurations for the pile lengths. For the shallower excavation (configuration 1), the primary piles were 13 m deep, while the secondary piles were 16 m long. For the deeper excavation (configuration 2), the primary piles were 16 m deep, while the secondary piles were 22 m long. Details of the primary and secondary pile lengths can be seen in Figure 2 and 3.

Figure 3 shows the front view of the excavation plan. The top of the slope gradually declines as it goes to the right, from +364.94 m to +358.28 m. The deepest cut was 11 m high, while the shallowest cut was 5 m high. At the level marked as GFL there was 8.65 m wide bench before another 4 m to 4.7 m was cut, reaching the road level. The road level was also the base level of the house to be built. This bench was retained by a gravity wall, and part of the house's second level will sit on this bench (please refer to Figure 4 for cross-sectional view at the deepest excavation depth).

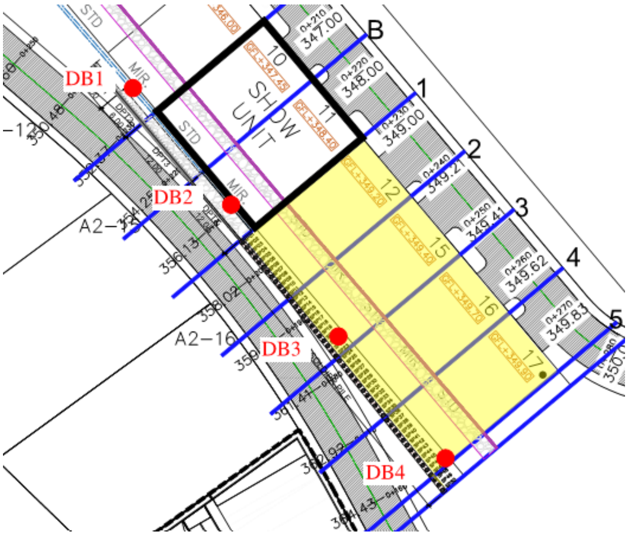

Figure 1. Plan view of the excavated site and soil investigation locations (area highlighted yellow is the excavated area)

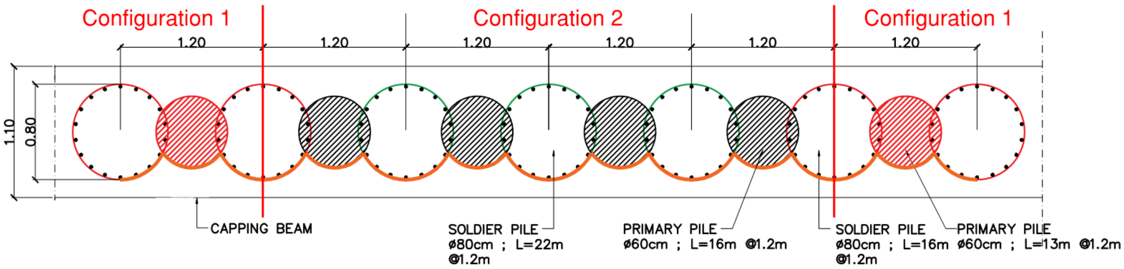

Figure 2. Primary and secondary pile configuration

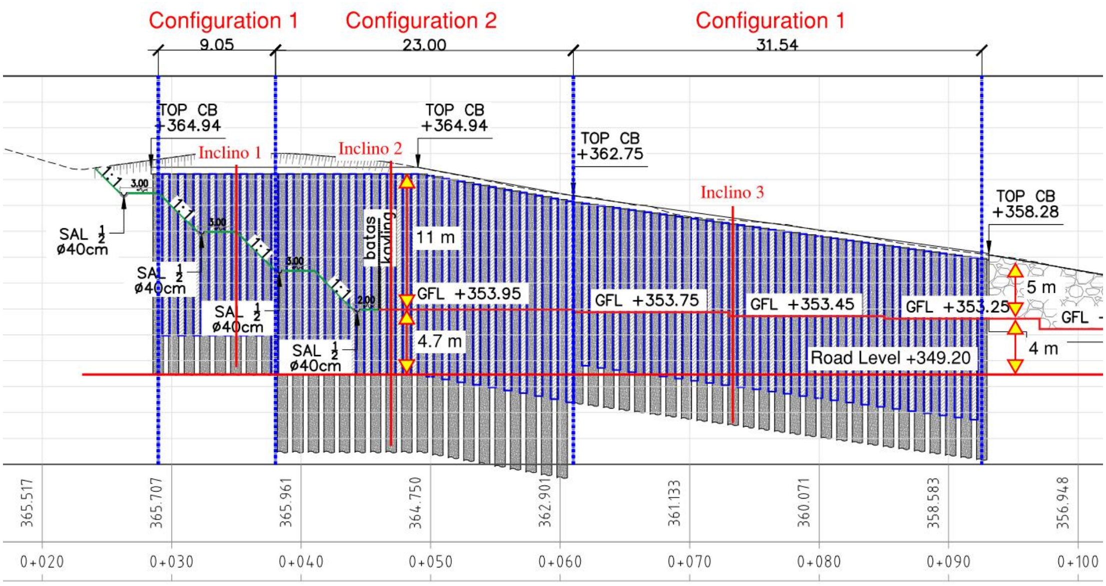

Figure 3. Front view of the excavation and secant pile configuration and depth (primary piles are in blue)

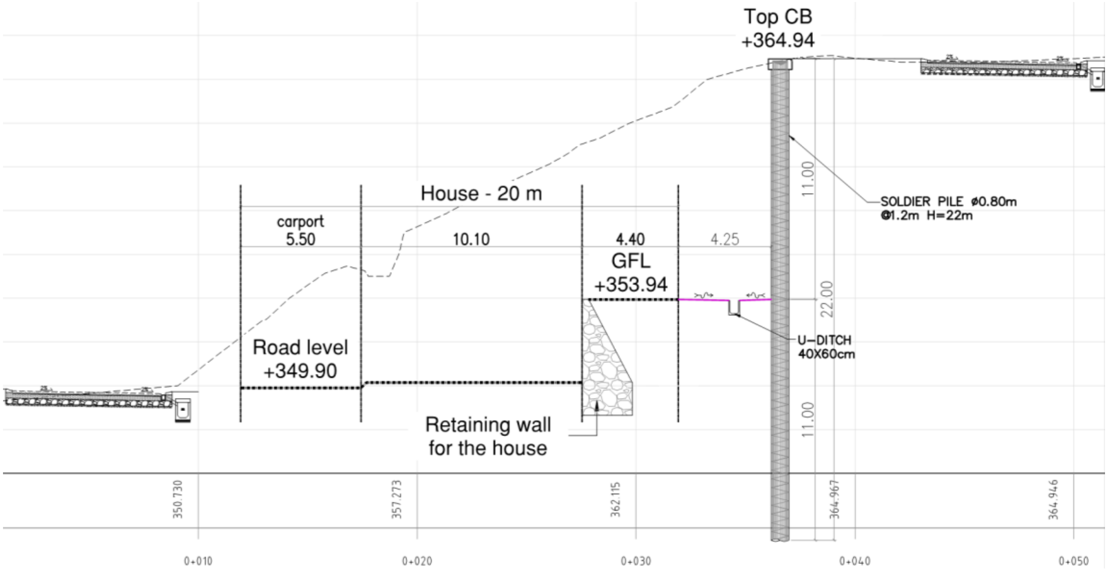

Figure 4. Cross section at the deepest excavation depth (cross section at inclino 2 in Figure 3)

Figure 5 shows a photograph of the excavation to give a better visualization of the excavation and retaining system. The location of the three inclinometers used for monitoring is also shown in the figure. The inclinometers were installed till 1 m below the base of the pile (i.e., 15 m for inclinometer 1 and 3; 21 m for inclinometer 2). During excavation, inclinometer readings were conducted twice a day until the completion of excavation after twelve days. Aerial and front view photographs of the excavation at different stages are shown in Figure 6. By day 12, the end of the daily inclinometer monitoring, majority of the excavation was already done.

Figure 7 shows the NSPT data for the four boreholes conducted. With the exception of DB1 which encountered boulder at 3 m depth, the other 3 boreholes revealed that the soil stratigraphy can be separated into three distinct layers. Medium silt (average NSPT ≈ 5) to a depth of 6 m, underlain by 4 m of stiff silt (average NSPT ≈ 12), and a hard silt layer (NSPT > 60) extending beyond the investigation depth. Undisturbed samples were also taken during the borehole investigations. Triaxial consolidated undrained (CU) tests were conducted on the undisturbed samples. The effective friction angle varied from 20.5° to 27.5°, while the effective cohesion varied from 26.5 kPa to 34 kPa. The effective friction angle and cohesion used for design was 23° and 30 kPa, respectively.

Figure 5. Photograph of the excavation and location of inclinometers

Figure 6. Photographs of the excavation at different stages

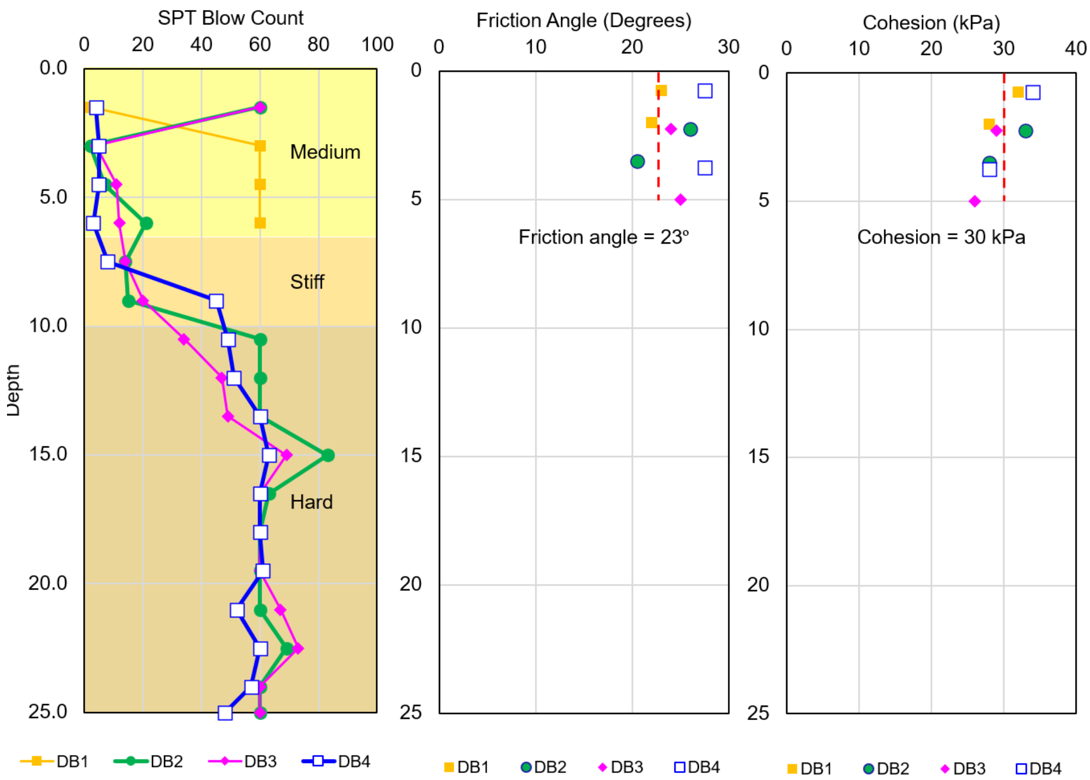

Figure 7. Soil investigation and effective shear strength obtained from triaxial CU (consolidated undrained)

2.2 Finite Element Modelling

The excavation was modelled using PLAXIS 2D finite element software. Figure 8 shows the slope model before the excavation was initiated. The model follows the typical boundary conditions, i.e., horizontal fixity at the left and right boundary; horizontal and vertical fixity at the bottom boundary.

Since the soil investigation was only conducted on the crest of the slope, along the secant pile, the soil layering was assumed to follow the contour of the slope. The ground water table was modelled as linear at the base of retaining wall. The excavation was done in six construction steps. In the first four construction steps, the excavation was done in 2 m intervals, while the fifth and final excavation was 3 m and 4 m, respectively. Figure 9 shows the final excavation step, prior to the construction of retaining wall for the house.

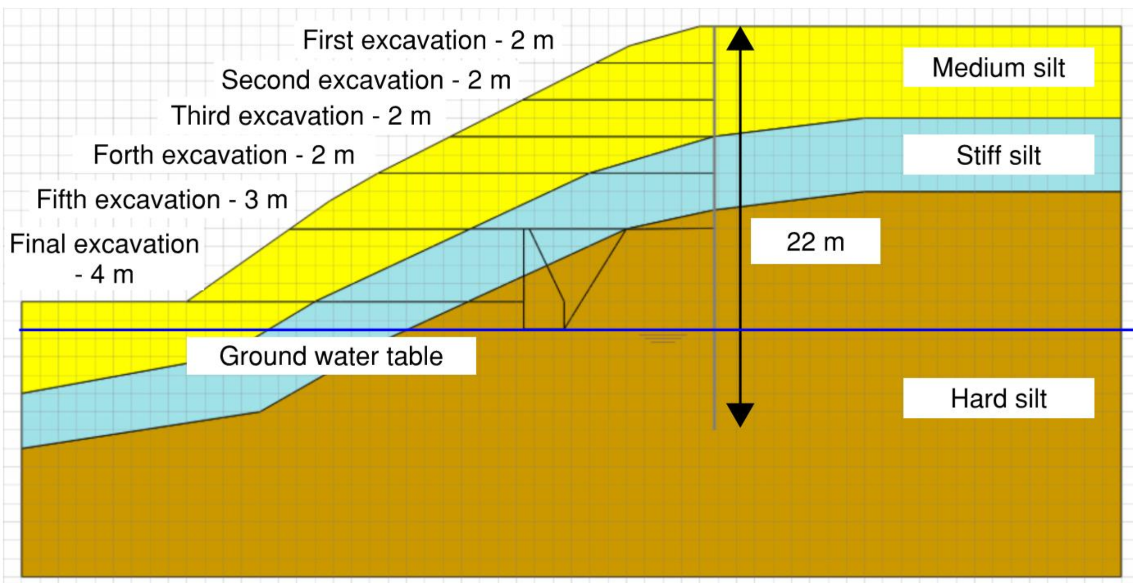

Figure 8. Finite element model before excavation

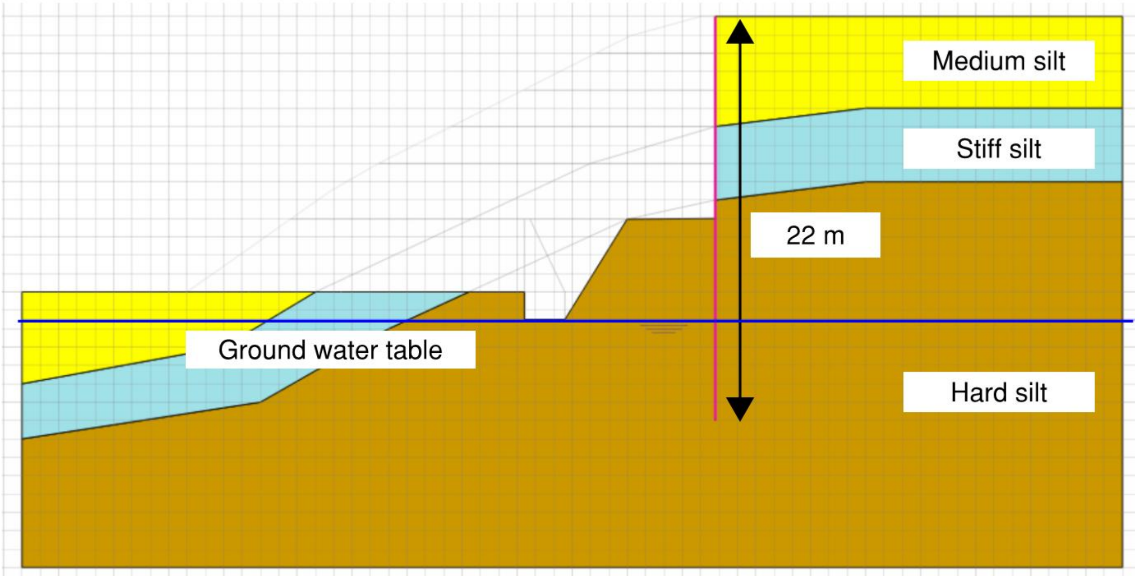

Figure 9. Finite element model after end of excavation

As aforementioned, the soil was separated into three layers. The soil was modelled using hardening soil model. The Young's modulus (E) was estimated using the equation proposed by Jones and Rust (1989), shown in equation 1.

$$E\ (\text{MPa}) = 1.6\ NSPT \quad (1)$$

For the unloading reloading modulus (E_{ur}), the recommended value in the PLAXIS manual is 2 to 5 times the loading modulus. For this analysis, a four times multiplier was used. Table 1 summarizes the soil parameters used for analysis.

Table 2. Soil parameters used for analysis

Name	γ_{unsat} kN/m^3	γ_{sat} kN/m^3	E_{50} kN/m^2	E_{ur} kN/m^2	c' kPa	ϕ' °
Medium silt	16.0	17.0	8000	32000	30	23
Stiff silt	17.0	18.0	19200	76800	40	28
Hard silt	20.0	20.5	96000	396000	50	35

The secant pile was modelled using plate element. Full contact, i.e., no interface or interface factor = 1.0 was assumed. It was also assumed that the primary pile did not contribute to the axial rigidity (*EA*) or flexural rigidity (*EI*) of the secant pile. The concrete used was of K300 grade, which gives a cylindrical compressive strength (f_c') of 24 MPa. For bored pile, it is recommended not to use the full concrete strength as some air maybe trapped during tremie concreting. Xing et al. (2003) recommends using only 80% of the concrete strength, yielding a compressive strength (f_c') of 19.2 MPa. Using the ACI 318 formula:

$$E_c = 4700\sqrt{f_c'}\ (\text{in MPa}) \quad (1)$$

Based on the formula, the modulus of concrete is 20.6 GPa. To take into account the contribution of reinforcement cage, the modulus of concrete is taken as 21 GPa. Giving an axial rigidity (*EA*) of 10.56 x 10^6 kN and flexural rigidity (*EI*) of 4.22 x 10^5 kN.m^2. As plate element does not allow for pile spacing input, the axial rigidity per meter run and flexural rigidity per meter run have to be manually divided by the pile spacing of 1.2 m. Hence, the input parameters for the plate element were: *EA* = 8.8 x 10^6 kN/m and *EI* = 3.52 x 10^5 kN.m^2/m.

3 RESULTS

Figure 10 shows the deformed mesh of the slope at the end of excavation. It can be seen that at the road level, some heave (less than 1 cm) can be observed due to the significant unloading from excavation. As expected, the secant pile moved forward due to the excavation, with the maximum displacement occurring at the top of the pile. The predicted movement was 2.3 cm; this meets the recommended maximum allowable deformation criterion of 0.5% H_e. H_e is the excavation height = 11 m, giving the allowable deformation = 5.5 cm. The soil at the back of the secant pile experienced settlement of less than 1 cm.

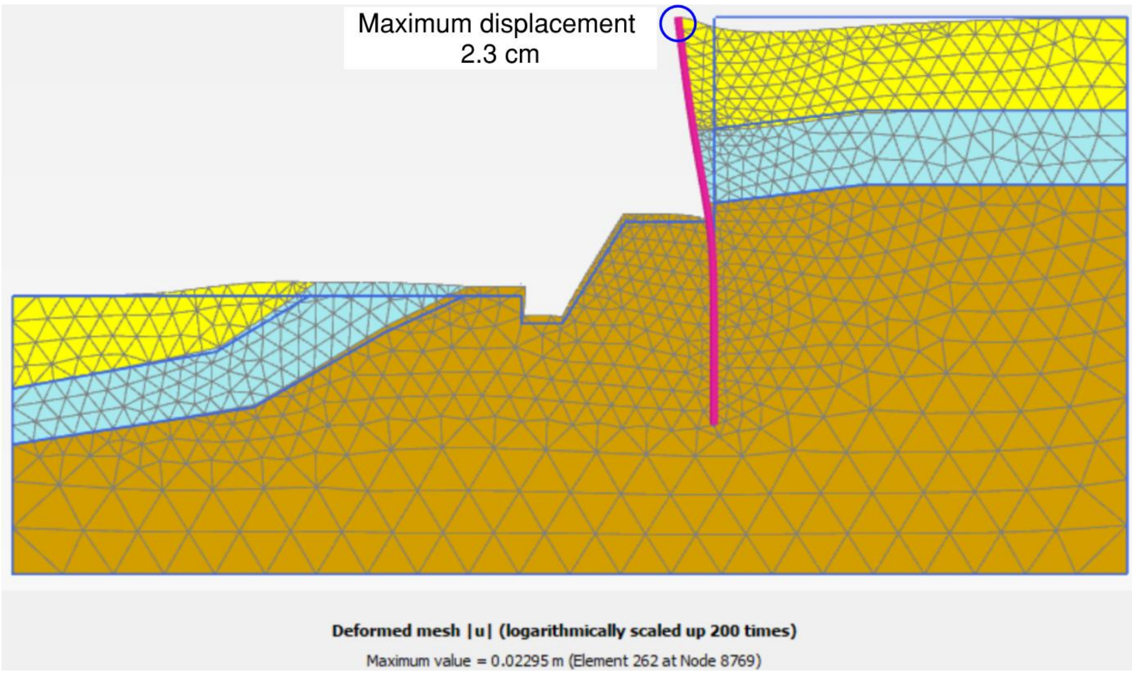

Figure 10. Deformed mesh at the end of excavation (magnified 200 times)

The inclinometer base line (initial) reading was taken 2 months prior the excavation. As aforementioned, the inclinometers were installed up to 1 m above the pile base, i.e., for inclinometer 1 and 3 (INC 01 and 02), they were installed to 15 m depth, while for inclinometer 2 (INC 02), it was installed to 21 m depth. Unfortunately, due to clogging (likely from too much sealant), inclinometer 2 could only reach 15 m depth, while inclinometer 3 reached 11 m depth. Nevertheless, it should not affect the rest of the reading as no movement below those depths were expected based on the prediction. The inclinometer device used has an accuracy of ± 0.08 mm per 0.5 m reading, giving an accuracy of ± 2.4 mm for 15 m length of readings.

Figure 11 shows the monitoring results for the three monitoring points. To prevent data clutter, only four out of the twenty-four available data points are plotted. Surprisingly, no noticeable movement occur throughout the monitored pile depth for all three inclinometers. For INC 01, there seems to be an inward movement recorded instead of outward. However, this might be due to errors during the base line/initial reading. The movement recorded stayed constant at -5 mm, meaning there was no movement recorded during excavation. For comparison, the predicted deformation profile from the finite element analysis was included for INC 02.

To better observe the evaluation of pile head movement during excavation, the twelve days pile head movement with time is plotted in Figure 11. Fluctuations can be observed for all three inclinometer data. With the exception of reading at day 3 for INC 03, the data fluctuated within ± 2 mm, which is within the expected ± 2.4 mm margin of error. Therefore, it can be safely concluded that the two-tiered excavation did not induce any movement on the secant pile for the monitored period. Long-term monitoring, especially during rainy seasons, should still be conducted.

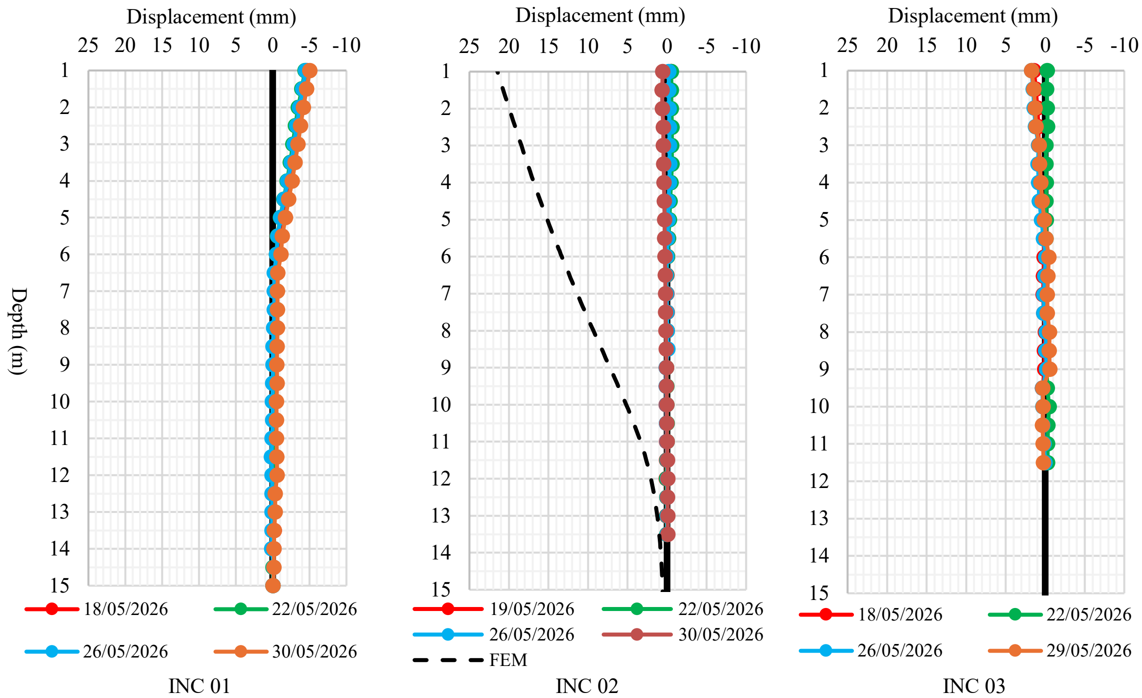

Figure 11. Displacement profile from inclinometer monitoring and finite element analysis for INC 02

Note: Positive displacement (leftward) is movement towards the excavation, while negative displacement (rightward) means the movement is into the retained soil.

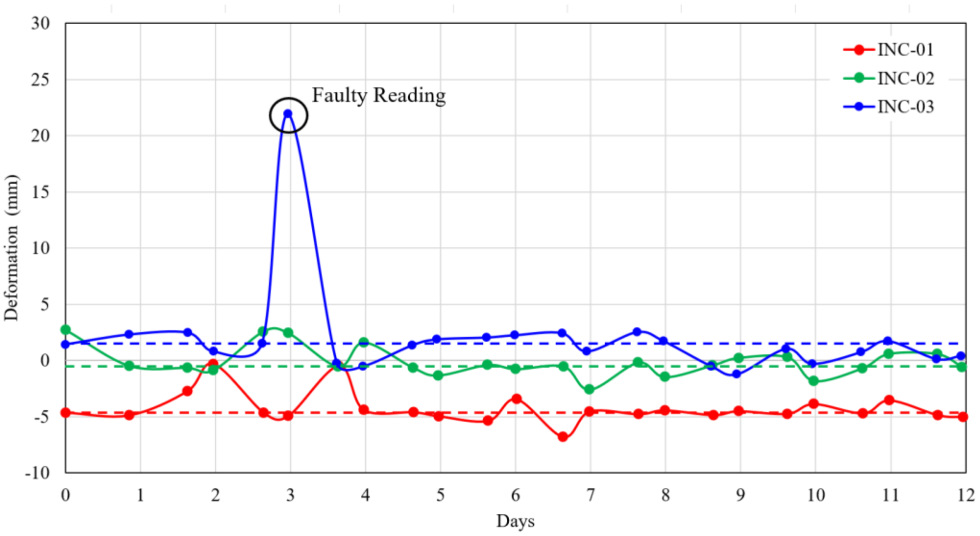

Figure 12. Displacement profile from inclinometer monitoring and finite element analysis for INC 02

4 DISCUSSION

The surprising zero movement on the secant pile wall despite the two-tiered excavation, totaling more than 15 m can be attributed to a few reasons. Firstly, the analysis conducted was 2D plane strain analysis, while in reality, the deepest excavation depth was only 5 m long and then gradually decreases. If the excavation was conducted in 3D finite element analysis, less deformation can be expected. Secondly, the soil parameters used was from saturated triaxial CU tests. Whereas, the soil on site was actually in unsaturated conditions. In fact, there was no rain during the 12 days excavation. Lower deformation can be expected if unsaturated parameters are to be used. However, it should be noted that using saturated parameters is an acceptable approach as Bogor can be very wet during rainy season (more than 300 mm per month for 6 months). Lastly, volcanic residual soils tend to show much higher shear strength than those captured by standard penetration values. Mirna et al. (2026) attributed this to the natural cementation within residual volcanic soil. The disturbance caused during standard penetration testing results in underrepresentation of the residual soil's true strength and stiffness.

5 CONCLUSIONS

Daily monitoring on a two-tiered excavation retained by secant pile wall across twelve days have successfully been conducted. Pre-excavation analysis predicted a maximum deformation of 2.3 cm. However, the monitoring results show no movement during the whole twelve days excavation duration. This could be due to 3-dimensional effects of excavation depth distribution, the site unsaturated soil conditions which cannot be captured by traditional triaxial CU, as well as SPT values under-representing the real shear strength and stiffness of residual volcanic soils.

DISCLAIMER

The authors declare no conflict of interest.

AVAILABILITY OF DATA AND MATERIALS

All data are available from the author upon reasonable request.

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