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HIDDEN DRAGLOAD: RISKS POSED BY NEGATIVE SKIN FRICTION TO PILE INTEGRITY IN SOFT SOILS

Heri Khoeri^{1*}, Badaruddin², Muhammad Aziz Komarudin³, Naufal Rafif Rizqullah⁴

¹ Department of Civil Engineering, Faculty of Engineering, Universitas Tama Jagakarsa, Jakarta, Indonesia

² Department of Civil and Environmental Engineering, Faculty of Engineering, Universitas Gadjah Mada, Yogyakarta, Indonesia

³ PT Hesa Laras Cemerlang, Jakarta, Indonesia, <https://hesa.co.id>

⁴ PT Stratalift Solusi Indonesia, Jakarta, Indonesia, <https://stratalift.co.id>

*¹[hkhoei@gmail.com](mailto:hkhoeri@gmail.com), ²badaruddin.badar79.1@gmail.com, ³kontak@hesa.co.id,

⁴naufalnjr18@gmail.com

Abstract

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Consolidation of soft soil surrounding pile foundations can generate negative skin friction and dragload that is not represented by the superstructure load. This study presents an integrated field, analytical, and numerical diagnosis of pile vulnerability at an operating industrial facility in Kutai Kartanegara, East Kalimantan, where 3.5 m of fill overlies 3–7 m of peat and soft clay. Sixth-year field observations indicated 0.50–0.80 m of ground settlement. Borehole and laboratory data, one-dimensional consolidation, dragload estimation, pile-length sensitivity from 18 to 28 m, and MIDAS SoilWorks modelling were combined. The estimated dragload was 330.7 kN, or 50.3% of the sustained structural load. Including dragload produced geotechnical utilization ratios of 1.334 and 1.013 for 18 m and 21 m piles. The 22 m pile was the minimum length satisfying the adopted axial geotechnical criterion, with $UR_g = 0.935$ and a 68.6 kN margin, but the maximum moment reached 86.86 kNm, or 95.9% of the adopted cracking moment. These results indicate limited geotechnical margin and structural vulnerability, not confirmed pile damage. Continued settlement monitoring and pile-integrity testing are required before intervention decisions.

Keywords: Consolidation; Dragload; Negative skin friction; Pile integrity; Soft soil

Introduction

Peat and soft clay exhibit high compressibility, low shear strength, and time-dependent deformation. Fill placement increases effective stress and initiates primary consolidation, while peat may continue to creep after most excess pore pressure has dissipated. Settlement is influenced by stress history, drainage, sample disturbance, fibre structure, and secondary compression; consequently, substantial deformation may continue long after construction (Cui et al., 2024; Peng et al., 2021, 2023, 2024; Tyurin & Nevzorov, 2017).

When the surrounding soil settles more than an embedded pile, the relative downward movement mobilizes negative skin friction. The resulting force is termed *dragload*, whereas the associated additional pile settlement is termed *downdrag*. Shaft resistance changes direction at the neutral plane, where pile and soil movements are compatible and the pile axial force generally reaches its maximum. Negative skin friction is governed by the compressibility and thickness of the settling layer, surcharge, consolidation degree, pile-head load, pile stiffness, penetration depth, toe response, and interface properties. Previous studies have demonstrated the influence of pile-head load, creep, pore-pressure dissipation, groundwater variation, surface loading, and pile geometry on dragload and neutral-plane position (Chai et al., 2023; Gen et al., 2022; Hong et al., 2016; Kim et al., 2018; Liang et al., 2023; Lu et al., 2024).

Pile-group behaviour introduces further uncertainty because spacing, pile position, cap restraint, and group-induced soil movement may cause unequal dragload and neutral-plane depths among individual piles (Altahrany & R. Elshehahwy, 2022; Kong, Zhou, & Yang, 2016; Kong, Zhou, Peng, et al., 2016; Mishra & Patra, 2018; Su et al., 2020). Recent foundation assessments similarly show that pile dimensions, penetration depth, group configuration, and pile-cap restraint significantly influence bearing capacity and settlement (Jinan et al., 2025; Priyono et al., 2025). However, these studies primarily evaluated conventional positive resistance and settlement rather than time-dependent dragload generated by consolidating soil. Accordingly, single-pile results provide only a screening basis unless pile-group interaction and compatible cap movement are explicitly modelled.

This study examines an operating palm-oil storage and distribution facility in Kutai Kartanegara, East Kalimantan, constructed on approximately 3.5 m of fill over 3–7 m of peat and soft clay. Estimated ground settlement in the sixth year ranged from 0.50 to 0.80 m. Several structures are supported by 400 × 400 mm precast concrete piles of varying lengths, while superstructure distress has raised concern about foundation deformation. Field studies integrating settlement observations, consolidation analysis, dragload, length-dependent capacity, and pile internal forces in an operating industrial facility remain limited. This study therefore identifies the settlement-controlling strata, estimates dragload relative to sustained structural load, evaluates pile lengths of 18–28 m, examines finite-element deformation and internal forces, and establishes a precautionary basis for integrity testing. Its contribution is an integrated diagnostic framework for interpreting pile vulnerability, rather than a new negative-skin-friction formulation.

Research Methods

Site and Data

The study examined an industrial facility in Kutai Kartanegara, East Kalimantan, constructed on approximately 3.5 m of fill over clay and peat. The interpreted soil profile extended to 28 m, with the most compressible zone at approximately 4–10 m depth, where N-SPT values were 1–2 and the peat compression index reached 2.752. The assessment used borehole logs, N-SPT results, laboratory parameters, structural reactions, pile geometry, MIDAS SoilWorks models, and sixth-year ground-settlement data of 0.50–0.80 m. The evaluated element was a 400 × 400 mm precast concrete pile with a design length of 22 m. Lengths of 18–28 m were examined to evaluate the influence of toe depth on positive resistance, dragload, and capacity margin. The principal inputs

are summarized in Table 1.

Table 1. Principal data used in the pile assessment

Parameter	Value
Fill thickness	≈3.5 m
Peat and soft-soil thickness	≈3–7 m
Measured settlement in year 6	0.50–0.80 m
Pile type	Square precast concrete pile
Pile cross-section	400 × 400 mm
Cross-sectional area	0.16 m ²
Pile perimeter	1.60 m
Pile self-weight	3.84 kN/m
Design pile length	22 m
Evaluated length range	18–28 m
Maximum sustained axial load	656.9 kN
Tank pressure in numerical model	≈137 kPa
Existing-design safety factor	2.0

Assessment Procedure

The assessment comprised stratigraphic interpretation, identification of the consolidating zone, calculation of positive pile resistance and dragload, pile-length sensitivity analysis, finite-element modelling, and integrity-risk screening. Analytical calculations were used to distinguish resistance and load components, whereas numerical results were used to evaluate soil–pile deformation compatibility and pile internal forces. The procedure is summarized in Figure 1.

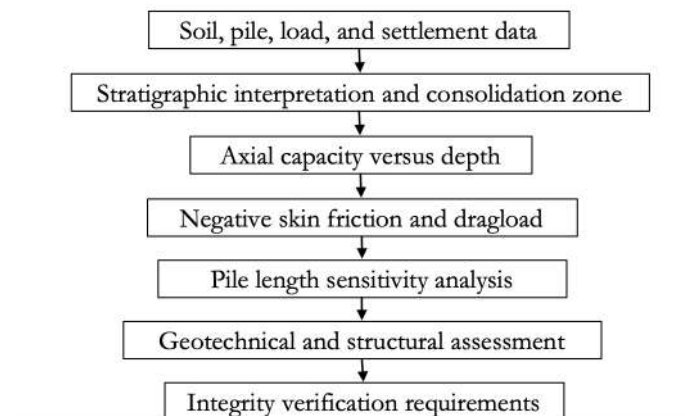


Figure 1. Workflow for evaluating negative skin friction and pile-integrity vulnerability

Axial Pile Capacity

Ultimate positive capacity (Q_u) was calculated as the sum of toe and shaft resistance:

$$Q_u = Q_p + Q_s$$

Positive shaft resistance in cohesive soil was obtained using the adhesion method:

$$Q_s = \sum_{i=1}^n \alpha_i c_{u,i} p \Delta L_i$$

where α_i is the layer-specific adhesion factor, $c_{u,i}$ is undrained shear strength, p is pile perimeter, and ΔL_i is the pile length in the layer. The calculation used the adhesion factors recorded

in the project pile-capacity worksheet, selected by soil consistency and strength; the values should be read together with the adopted c_u profile rather than treated as independently calibrated parameters. Because continuous laboratory measurements of undrained shear strength were unavailable, c_u was estimated using the project correlation $c_u = 6\text{NSPT}$ (kPa). This empirical correlation was checked against soil descriptions and available laboratory data but remains uncertain for organic and highly compressible soils.

$$c_u = 6\text{NSPT}$$

The approach follows SNI 8460:2017 and conventional pile-design practice (BSN, 2017; Hardiyatmo, 2019). Toe resistance and allowable positive capacity were:

$$Q_p = N_c c_{u,p} A_p$$

$$Q_a = \frac{Q_p + Q_s}{FS}$$

where $N_c = 9$, A_p is the pile-toe area, and $FS = 2.0$, consistent with the existing design basis. The representative c_u at the toe was evaluated over its influence zone.

Negative Skin Friction and Dragload

Negative skin friction was assigned to pile segments where downward soil movement exceeded pile movement. Unit negative shaft resistance was calculated using:

$$\tau_{nsf,i} = \alpha_i c_{u,i}$$

and cumulative dragload (Q_{drag}) was obtained from:

$$Q_{drag}(z) = \sum_{i=1}^n \tau_{nsf,i} p \Delta L_i$$

Dragload was treated as an additional sustained action rather than being subtracted from positive resistance (Ashour & Abbas, 2021; Hong et al., 2016; Liang et al., 2023; Wang et al., 2024). Pile adequacy was therefore checked using:

$$P_s + Q_{drag} \leq Q_a$$

where P_s is the sustained pile-head load (Jiang et al., 2023; Wu et al., 2022). Axial force along the pile was represented as:

$$N(z) = P_s + Q_{drag}(z) - R_s(z)$$

where $N(z)$ is axial force and $R_s(z)$ is mobilized positive resistance above depth z . The region of maximum axial force was used to approximate the neutral-plane location because an instrumented axial-force profile was unavailable.

The dragload ratio $R_D = Q_{drag}/P_s$ was used to indicate the proportion of pile compression that developed from ground movement. Geotechnical utilization was calculated using:

$$UR_g = (P_s + Q_{drag})/Q_a$$

A pile was classified as adequate when $UR_g \leq 1.0$. Values approaching unity were interpreted as limited margins that require consideration of parameter uncertainty, spatial variability, pile installation, and actual pile condition.

Finite-Element Modelling

Finite-element analysis used MIDAS SoilWorks with a 28 m Mohr–Coulomb profile, restrained base and lateral boundaries, and coupled pile–soil elements. Investigation-based stiffness ranged from $E = 1,000$ kPa for peat ($\nu = 0.45$) to 46,800 kPa for deeper layers. A 137 kPa tank pressure was applied, and results were compared with sixth-year settlement data; pile-force

measurements were unavailable for calibration.

Performance and Integrity Assessment

Performance was assessed through geotechnical utilization (UR_g), soil–pile settlement compatibility, and structural actions. Cracking vulnerability was expressed as:

$$UR_M = M_{max}/M_{cr}$$

The adopted $M_{cr} = 90.61$ kNm was obtained from existing design documents but could not be independently reconstructed because complete reinforcement, prestress, splice, and material records were unavailable. Consequently, URM is a cracking-vulnerability indicator rather than an ultimate axial-moment capacity check. The assessment used one sixth-year settlement dataset, an empirical N-SPT– c_u correlation, constant dragload across pile lengths, and single-pile calculations without explicit group interaction or cap restraint. Accordingly, the results indicate vulnerability, while integrity loss requires pile-integrity or dynamic testing.

Research Results and Discussion

Stratigraphy and Settlement Development

The critical 4–10 m zone comprised highly compressible very soft clay and peat with N-SPT values of 1–2, whereas N-SPT increased to 13–47 below 10 m. This stiffness contrast controlled settlement and the potential negative-friction zone. The adopted parameters are presented in

Table 2.

Table 2. Interpreted soil profile and principal analysis parameters

Depth (m)	Soil description	N-SPT	c_u (kPa)	E (kPa)	C_c
0–2	Very soft clay/fill	1–10	6–60	12,000	0.250
2–4	Stiff clay/fill	10	60	12,000	0.025
4–7	Very soft clay	2	12	2,400	0.125
7–10	Peat	1	6	1,000	2.752
10–12	Stiff clay	13	78	15,600	0.019
12–16	Very stiff clay	23	138	27,600	0.011
16–28	Hard clay	39	234	46,800	0.006

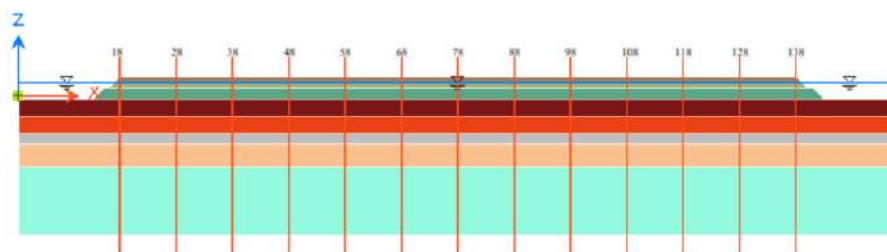


Figure 1. One-dimensional consolidation model of the fill and subsoil in MIDAS SoilWorks

shows the consolidation model, in which the highly compressible peat at 4–10 m controlled settlement. MIDAS SoilWorks was used to assess continuing soil–pile relative movement, considering drainage, compressibility, and secondary compression (Khoeri, Isvara, et al., 2024; Peng et al., 2021, 2024; Yuswandono et al., 2020).

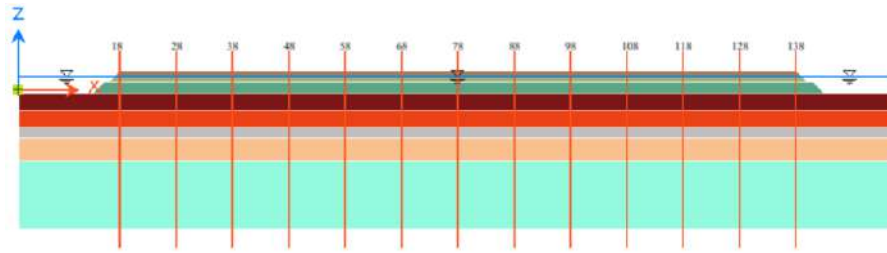


Figure 1. One-dimensional consolidation model of the fill and subsoil in MIDAS SoilWorks

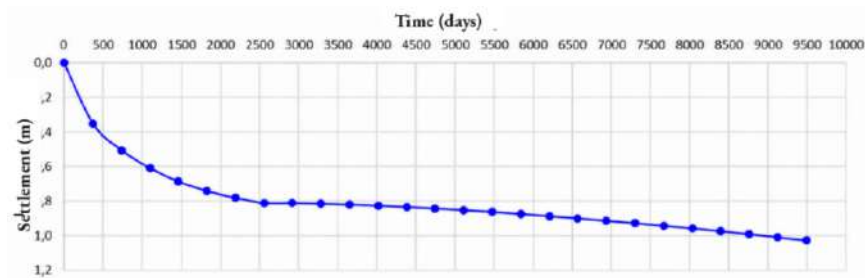


Figure 2. Predicted settlement development with time

Predicted settlement reached 0.608 m in year 3 and 0.809 m in year 6, with a decreasing rate as primary consolidation exceeded 90% (Figure 3). Residual deformation may nevertheless continue through remaining primary consolidation and secondary compression (Cui et al., 2024; Tyurin & Nevzorov, 2017). The year-6 prediction was consistent in magnitude with the observed range of 0.50–0.80 m, supporting substantial soil–pile relative movement and potential dragload development (Hong et al., 2016; Kim et al., 2018). However, one sixth-year dataset cannot validate the settlement history or represent spatial variations in soil, loading, drainage, and groundwater conditions; continued elevation monitoring is therefore required.

Evidence of Negative Skin Friction Mobilization

Finite-element pile-head settlement was 23.95 mm, compared with 500–800 mm of ground settlement, producing a relative displacement of approximately 476–776 mm. This substantial difference indicates downward soil movement relative to the pile and a high likelihood of negative skin friction, but does not prove pile damage.

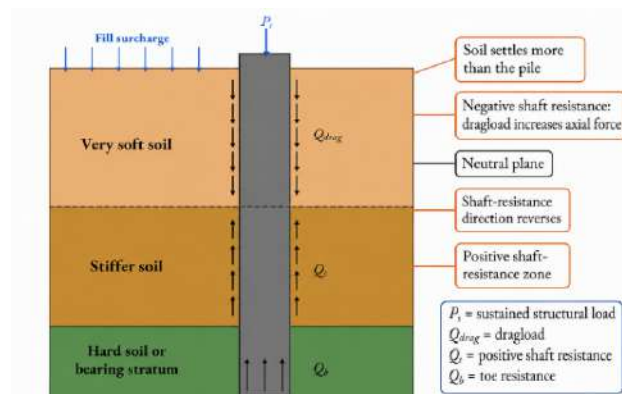


Figure 3. Negative skin friction, dragload, and shaft-resistance reversal along a pile

As illustrated in Figure 3, downward soil movement above the neutral plane generates dragload, whereas shaft resistance below it becomes positive. The neutral-plane position is governed by compatibility among settlement, sustained load, shaft resistance, and toe resistance rather than simply by the base of the soft layer (Coffman et al., 2024; Tan & Fellenius, 2016). The calculated dragload was 330.7 kN, equivalent to 50.3% of the sustained structural load of 656.9 kN. This additional force develops internally as the soil consolidates and is not reflected in the superstructure reaction. The result is consistent with previous soil–pile interaction studies (Chiou & Wei, 2021; Liang et al., 2023; Lin et al., 2021; Lu et al., 2024), but remains a single-pile screening estimate because pile-group interaction was not explicitly modelled.

Effect of Dragload on Pile-Length Adequacy

Positive resistance increased as the pile toe entered stronger strata. A constant $Q_{drag} = 330.7$ kN was added to the demand because every evaluated pile crossed the same approximately 10 m settling zone and the screening calculation assumed full mobilization within that zone. This permits a consistent length comparison but does not reproduce length-dependent mobilization or pile-group effects (Table 3).

Table 3. Effect of pile length on capacity with negative skin friction active

L_{pile} (m)	Q_a (kN)	P_s (kN)	Q_{drag} (kN)	$P_s + Q_{drag}$ (kN)	UR_g	Margin (kN)	Status
18	740.4	656.9	330.7	987.6	1.334	-247.2	Inadequate
21	974.6	656.9	330.7	987.6	1.013	-13.0	Inadequate
22	1,056.2	656.9	330.7	987.6	0.935	68.6	limited margin
23	1,198.3	656.9	330.7	987.6	0.824	210.7	Adequate
28	1,760.4	656.9	330.7	987.6	0.561	772.8	Adequate

Including dragload increased sustained demand from 656.9 to 987.6 kN. Piles of 18 m and 21 m exceeded the adopted axial criterion, with $UR_g = 1.334$ and 1.013. The 22 m pile was the minimum length meeting that criterion, with $UR_g = 0.935$ and a 68.6 kN margin; lengths of 23 m and 28 m reduced utilization to 0.824 and 0.561. Figure 5 places the threshold between 21 m and 22 m, but the 22 m margin is only 6.5% and is sensitive to empirical strength, spatial variability, installation effects, and the constant-dragload (Ashour & Abbas, 2021; Bartz & Blatz, 2020, 2023).

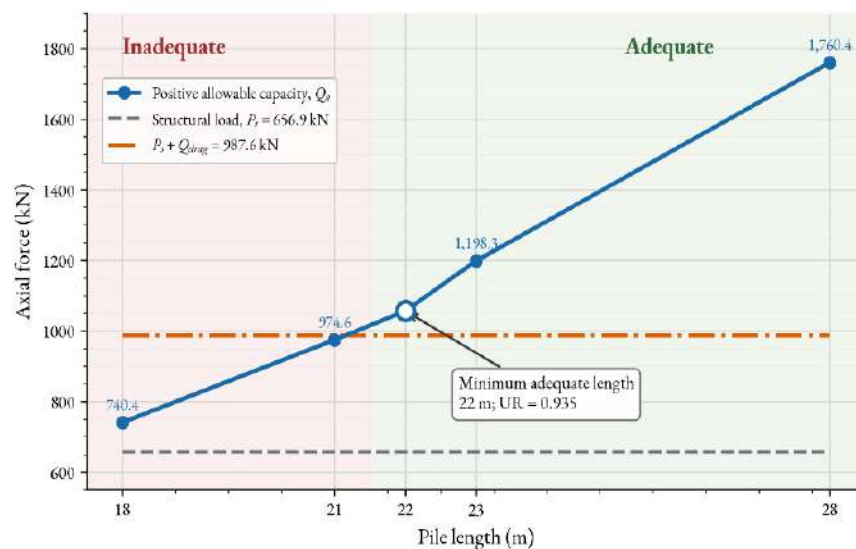


Figure 4. Positive allowable capacity and total dragload-inclusive demand versus pile length

Sensitivity of the 22 m Pile

A one-at-a-time $\pm 20\%$ screening variation was applied to positive allowable capacity and dragload. Reducing Q_a by 20% increased UR_g to 1.169, while increasing Q_{drag} by 20% increased UR_g to 0.998. The combined adverse case produced $UR_g = 1.247$ (Table 4), confirming that the baseline classification is not robust to plausible parameter uncertainty.

Table 4. Sensitivity of geotechnical utilization for the 22 m pile

Scenario	Q_a (kN)	Q_{drag} (kN)	UR_g	Interpretation
Q_a -20%	845.0	330.7	1.169	Exceeds criterion
Q_{drag} +20%	1,056.2	396.8	0.998	At threshold
Baseline	1,056.2	330.7	0.935	Limited margin
Q_{drag} -20%	1,056.2	264.6	0.872	Below criterion
Q_a +20%	1,267.4	330.7	0.779	Below criterion
Combined adverse	845.0	396.8	1.247	Exceeds criterion

Table 5. Performance indicators for the 22 m design pile

Indicator	Value
Maximum sustained structural load, P_s	656.9 kN
Estimated dragload, Q_{drag}	330.7 kN
Dragload-to-structural-load ratio	50.3%
Total sustained demand, $P_s + Q_{drag}$	987.6 kN
Positive allowable capacity, Q_a	1,056.2 kN
Geotechnical utilization, UR_g	0.935
Remaining axial margin	68.6 kN
Pile-head settlement	23.95 mm
Maximum bending moment, M_{max}	86.86 kNm
Adopted cracking moment, M_{cr}	90.61 kNm
Cracking-moment ratio, UR_M	0.959

Table 5 shows that, despite geotechnical adequacy, the 22 m pile remains structurally vulnerable because dragload equals 50.3% of the structural load, soil–pile displacement is substantial, and $UR_M=0.959$. These indicators do not confirm pile cracking or integrity loss. The maximum axial force may occur near the neutral plane, whereas group response depends on pile spacing, position, and cap restraint.

Finite-Element Pile Response

Figure 5 shows the finite-element model used to evaluate sixth-year soil deformation and pile internal forces under tank loading.

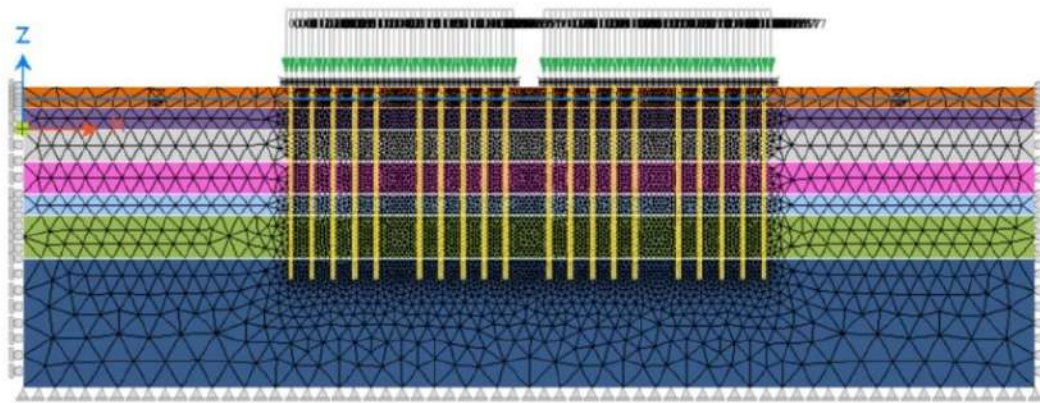


Figure 5. Finite-element model of the pile foundation and layered soil in MIDAS SoilWorks

Figures 7 and 8 show vertical soil–pile incompatibility that mobilizes negative skin friction and horizontal movement that may induce additional bending near stiffness transitions, splices, and pile-head restraints.

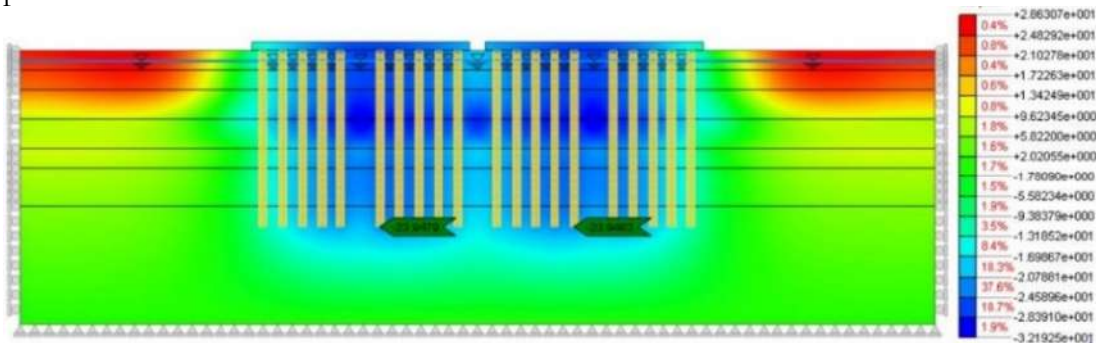


Figure 6. Vertical-displacement contour of the soil and pile foundation (mm)

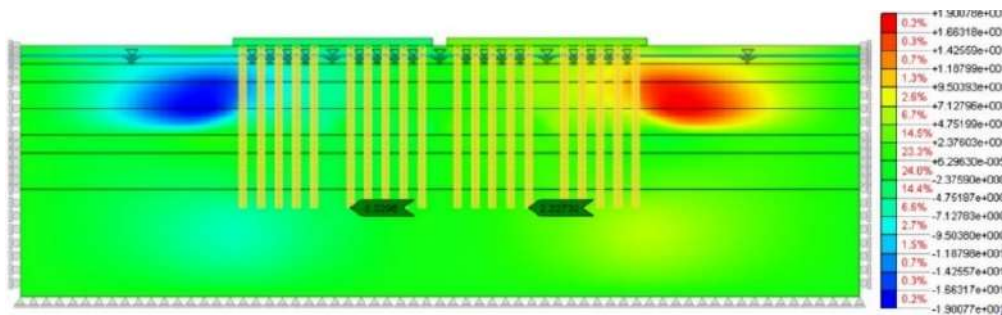


Figure 7. Horizontal-displacement contour of the soil and pile foundation (mm)

Figures 9–11 show force concentrations near the pile head and stiffness transitions. The maximum moment was 86.86 kNm, close to the adopted cracking moment of 90.61 kNm, indicating vulnerability to combined axial load, lateral deformation, and head restraint.

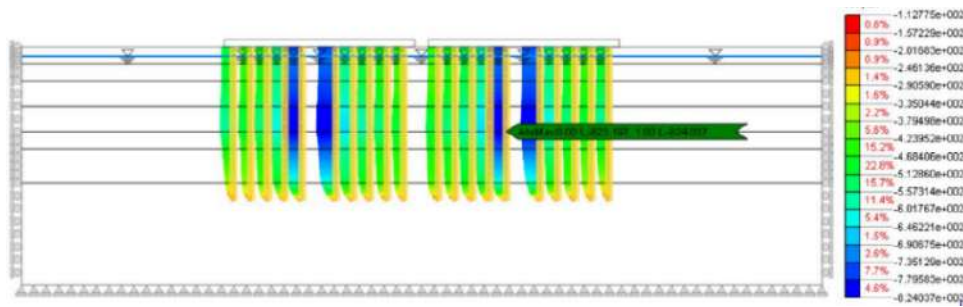


Figure 8. Axial-force distribution in the piles from finite-element analysis (kN)

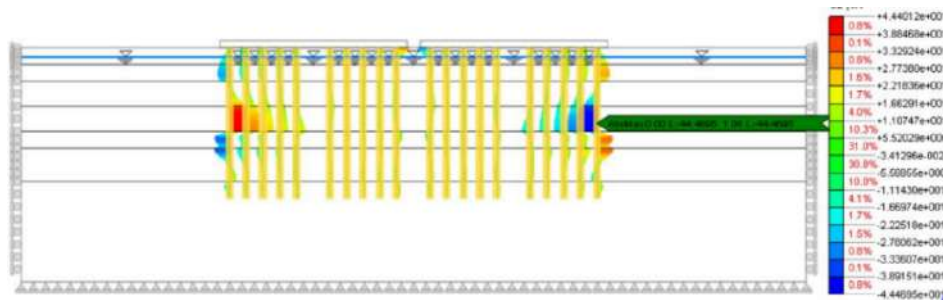


Figure 9. Shear-force distribution in the piles from finite-element analysis (kN)

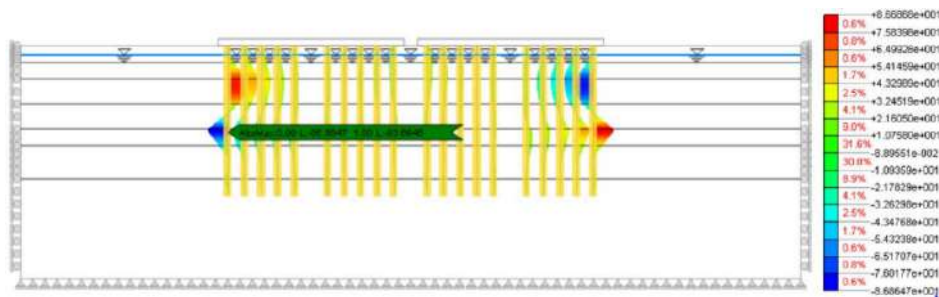


Figure 10. Bending-moment distribution in the piles from finite-element analysis (kN.m)

The calculated UR_M of 0.959 leaves only a 4.1% margin to the adopted cracking moment. However, incomplete pile and splice data prevent its use as an ultimate-capacity check or proof of cracking.

Implications for Pile Integrity and Engineering Practice

Ground settlement was 20–33 times the modelled pile-head settlement, dragload equalled about half the structural load, and the moment reached 95.9% of M_{cr} . Thus, geotechnical adequacy alone cannot confirm pile integrity. For new construction, negative-friction risk may be reduced through staged filling, preloading, ground improvement, interface treatment, or modified pile geometry (Feng et al., 2019; Kong, Zhou, Peng, et al., 2016; Shawky et al., 2022). Existing piles should first be screened using low-strain Pile Integrity Testing, followed by dynamic, parallel-seismic, or Cross-Hole Sonic Logging where anomalies are identified, with particular attention to precast-pile splices (G. S. Budi & Tanaya, 2022; Ertel et al., 2016; Liu et al., 2020, 2023; Ly et al., 2022).

The proposed sequence integrates identification of settling layers, dragload and capacity assessment, evaluation of combined pile actions, and field integrity verification.

Conclusion

Sixth-year field observations indicated 0.50–0.80 m of ground settlement, whereas the finite-element pile-head settlement was 23.95 mm, indicating strong potential for negative skin friction. The analytically estimated dragload was 330.7 kN, or 50.3% of the sustained structural load. Including dragload, the 22 m pile was the minimum length meeting the adopted axial geotechnical criterion ($UR_g = 0.935$; margin 68.6 kN), while shorter piles exceeded it. Sensitivity cases showed that a 20% reduction in capacity or a combined adverse variation would reverse this classification. The computed maximum moment was 86.86 kNm, or 95.9% of the adopted cracking moment; this indicates vulnerability, not confirmed damage. Benchmark-based monitoring and low-strain integrity testing are therefore recommended, followed by targeted dynamic or geophysical verification. Future work should include actual reinforcement and splice details, axial-moment interaction, spatial soil variability, and instrumented axial-force data.

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Declaration of Generative AI Use

During the preparation of this manuscript, the authors used ChatGPT by OpenAI to improve English-language readability, and manuscript organization. The authors subsequently reviewed and edited the content and take full responsibility for the final manuscript.

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