



The Piling in Glauconitic Sand (PIGS) JIP: insights from axial and lateral pile load testing

F. Pisanò*

Norwegian Geotechnical Institute, Boston, Massachusetts, USA

Z. Westgate

University of Massachusetts Amherst, Amherst, Massachusetts, USA

A. Rahim, C. Maldonado

Norwegian Geotechnical Institute, Houston, Texas, USA

V. Komurka

GRL Engineers, Cleveland, Ohio, USA

R. Beemer

Virginia Polytechnic Institute and State University, Blacksburg, Virginia, USA

(formerly University of Massachusetts Dartmouth, Dartmouth, Massachusetts, USA)

B. Stuyts

Ghent University, Ghent, Belgium

L. Hamre

DNV, Oslo, Norway

G. Eiksund

Norwegian University of Science and Technology, Trondheim, Norway

E. Liedtke

bp, Houston, Texas, USA (formerly Equinor, Stamford, Connecticut, USA)

Y. Perikleous

DNV, Copenhagen, Denmark (formerly Ørsted, Copenhagen, Denmark)

A. Ridgway-Hill

Ørsted, London, UK

J. De Sordi

Equinor, Stavanger, Norway

A. Roux

Attentive Energy, Paris, France

L. Jones

Avangrid, London, UK

P. Ghasemi

RWE, Hamburg, Germany

Federico.Pisano@ngi.no (corresponding author)

ABSTRACT: The Piling in Glauconitic Sand (PIGS) Joint Industry Project (JIP) was established in 2021 to investigate the geotechnical behavior of glauconitic sands in relation to pile installation and operational performance. The scope of work completed to date comprises (i) characterization of glauconite / glauconitic sands from onshore test sites in New Jersey and from offshore wind lease areas across the Atlantic OCS, (ii) impact driving of steel pipe piles through glauconitic sands, and (iii) axial and lateral load testing of these piles. This paper presents insights derived from field data acquired during pile load

testing at Search Farm. Two axial monotonic load tests were conducted on below-grade-instrumented closed-end piles, each with a diameter of 0.32 m. These piles were driven to depths of 20.1 m and 14.9 m below ground surface before being subjected to compressive and tensile loading, respectively. Four larger piles of diameter 1.52 m were laterally loaded after being driven to a target depth of 15.0 m. Analysis of pile load test data supports understanding of load-transfer mechanisms. The crushed glauconite zone created around the pile during driving is confirmed to cause a transition from coarse- to fine-grained behavior, affecting axial pile geotechnical response. Difficulties in correlating traditional interpretations of CPT results to post-driving pile response to loading are also highlighted.

Keywords: US offshore wind; US energy market; field testing; glauconite sand; pile loading

1 INTRODUCTION

Offshore wind development in the U.S. has been focusing primarily on the North Atlantic and Mid-Atlantic Outer Continental Shelf (OCS). These regions are chosen due to their proximity to high energy demand areas, suitable shallow water depths, and strong wind conditions. The Atlantic OCS, designated as a 'frontier' region by the Bureau of Ocean Energy Management (BOEM), contains glauconite-rich sediments found in multiple lease areas. Glauconite is a friable green to black iron-potassium phyllosilicate mineral. This mineral exhibits unique behavior under shearing: particle sizes are reduced, and this can transform glauconite-rich layers from a coarse-grained to a fine-grained soil. This transformation, with associated changes in stress conditions in the soil, can result in high soil resistance to driving (SRD) and can potentially lead to premature pile installation refusal (DeGroot et al., 2023). To address these uncertainties, the Piling in Glauconitic Sand (PIGS) Joint Industry Project (JIP) was established in 2021 to better understand the behavior of glauconitic soils around piles through site investigations, field scale testing, and data interpretation. The goal is to develop a framework for assessing the impact of glauconite sand on offshore wind infrastructure development. While recent publications have addressed glauconite-related issues (Westgate et al., 2023; DeGroot et al., 2023) and provided early results from PIGS onshore pile installation tests (Westgate et al., 2024), this paper presents the findings of pile load tests conducted at the same site. These tests investigated the response of piles to axial and lateral loading, exploring the implications of driving-induced glauconite crushing.

2 PIGS JIP FIELD TESTING CAMPAIGN

2.1 Site description and characterization

The JIP test site – henceforth referred to as the Search Farm site – sits within this Atlantic glauconite belt and was selected based on previous scientific investigation work performed by Rutgers University.

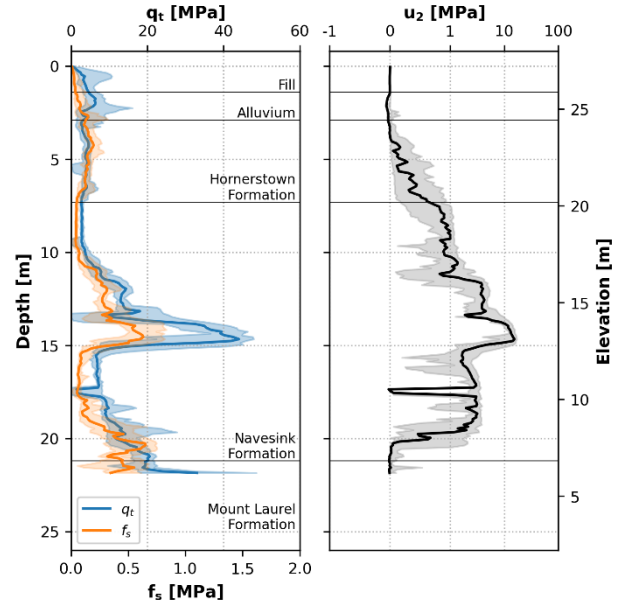


Figure 1. Median profiles of corrected cone resistance, sleeve friction, and pore water pressure at Search Farm site (shaded areas represent 10-90 percentile ranges)

The site is dominated by glauconite-rich soil, specifically the so-called Hornerstown and Navesink formations – details of the geological characteristics of these units are provided in Westgate et al. (2023). Site characterization included ground-penetrating radar, piezometric monitoring, standard penetration testing (SPT), seismic piezocone penetrometer testing (SCPTu), and novel sampling of crushed glauconite. Test results (see Figure 1) revealed a moderately dense, contractive glauconite sand deposit with a hard layer at 13-15 m depth, where cone tip resistance exceeded 40 MPa. The site slopes slightly southwest, with sandy fill in the upper 3 meters. The Hornerstown and Navesink Formations were found at depths of 3.2 m and 7.3 m, respectively. Glauconite content exceeds 95%, with particle density around 3.00 Mg/m³. CPT data show high tip resistance (q_t), and sleeve friction (f_s) ranging up to nearly 50 MPa and 1 MPa, respectively, with significant variation within the Navesink Formation. The site exhibits mostly positive excess pore pressure, exceeding 14 MPa in the hardest Navesink layer but occasionally reducing to negative values in

both the alluvium and glauconite layers. As noted by Westgate et al. (2024), soil behavior type (SBT) index (I_c) values are generally below 2.6, but the interpreted soil types differ between the Robertson Q_{in} - F_r chart (Robertson, 2016) and the Schneider Q_t - B_q chart (Schneider et al., 2008), emphasizing the need for careful interpretation of cone data.

2.2 Test piles and instrumentation

A total of nine steel pipe piles were used in the PIGS pile testing campaign (Table 1). Five smaller diameter piles (S1 through S5), featuring an outer diameter (OD) of ~ 0.32 m, were driven to depths between ~ 15 to 20 m. The S3 pile was open-ended and included an internal shoe, which was 9.5 mm thick and 10 cm long. Two of the closed-end S piles included external shoes, one with a single shoe at the toe of the same dimensions (Pile S4), and one with multiple shoes spaced 4.6 m apart along the pile length (Pile S5). Four larger-diameter piles (L1 through L4, OD of ~ 1.5 m) were driven to ~ 15 m depth, for a length-to-diameter ratio of 10. All four piles had internal shoes 25 mm thick and 10 cm long. Two of the piles (L2 and L4) included external shoes of the same dimensions to investigate reductions in shaft friction.

Three to four weeks after impact driving, Piles S1 and S2 were loaded monotonically to their axial peak resistances under compression and tension, respectively. Two types of strain sensors were installed on Pile S1, namely vibrating-wire “sister-bar” strainmeters and fiber optic sensors of the Fiber Bragg Grating (FBG) type; for Pile S2, only sister-bar strainmeters were used.

The L piles were subjected to post-installation lateral loading tests. Such tests were performed (three to four weeks after installation) on pairs of piles. The tests were configured to use a pair of hydraulic jacks acting in extension to pull the pile pairs together, using each other as a reaction for the opposing pile. Four types of sensors were deployed to measure pile deformations, namely (i) fiber optic sensors of the FBG type, (ii) Linear Variable Differential Transducers (LVDTs), (iii) Shape Acceleration Array (SAA) sensors, and (iv) vibrating-wire tiltmeters. Both tests’ setup provided for determining test loads three ways: (i) jack pressure (two identical manufacturer and model hydraulic jacks connected to a common manifold), (ii) four load cells (one associated with each tension threadbar which transferred loads between the test piles), and (iii) two strain gauges mounted diametrically opposed on machined-smooth sections of each of the four threadbars and on one of the bare threadbar sections.

Table 1. Overview of PIGS test pile geometry

Pile ID	Pile embedment depth	Pile diameter	Wall thickness	End condition
	m	m	mm	
L1	15	1.52	25.4	open
L2	15	1.52	25.4	open
L3	15	1.52	25.4	open
L4	15	1.52	25.4	open
S1	20.1	0.324	9.5	closed
S2	14.9	0.324	9.5	closed
S3	15.2	0.324	9.5	open
S4	20	0.324	9.5	closed
S5	20.3	0.324	9.5	closed

2.3 Installation tests

Pile driving was performed using a Junttan HHK9S hydraulic hammer for the S piles and a Junttan HHK20S hydraulic hammer for the L piles. Installation pauses and redrives were implemented into the installation program, to enable investigation of set-up effects – see Westgate et al. (2024).

During pile driving, Pile S3 partially plugged, while all 1.5-m-diameter piles were fully cored. Dynamic load testing was conducted using pairs of accelerometers on each pile head. Stroke height was adjusted to keep blow counts between 20 and 40 per quarter meter.

Overall, SRD was shown to be predicted more reliably when modeling the crushed glauconite sand as clay. Assuming sand model behavior exclusively was found to underestimate the SRD by a factor greater than 2. This is broadly consistent with other back-analyses in glauconite sands (Perikleous et al. 2023).

3 AXIAL LOADING

The test setup illustrated in Figure 2a was employed for the axial compressive test on Pile S1, 34 days after installation. The measured axial load-displacement response in Figure 2b showed a global nearly linear response up to 40% of its peak resistance (~ 5 MN at 32 mm displacement). Following this peak, the pile experienced geotechnical failure, evidenced by plunging and a “softening” response likely due to reduced shaft friction – which would be consistent with API softening t - z responses for clay. The unloading phase showed a nearly parallel linear response, suggesting no significant changes in soil properties under monotonic loading.

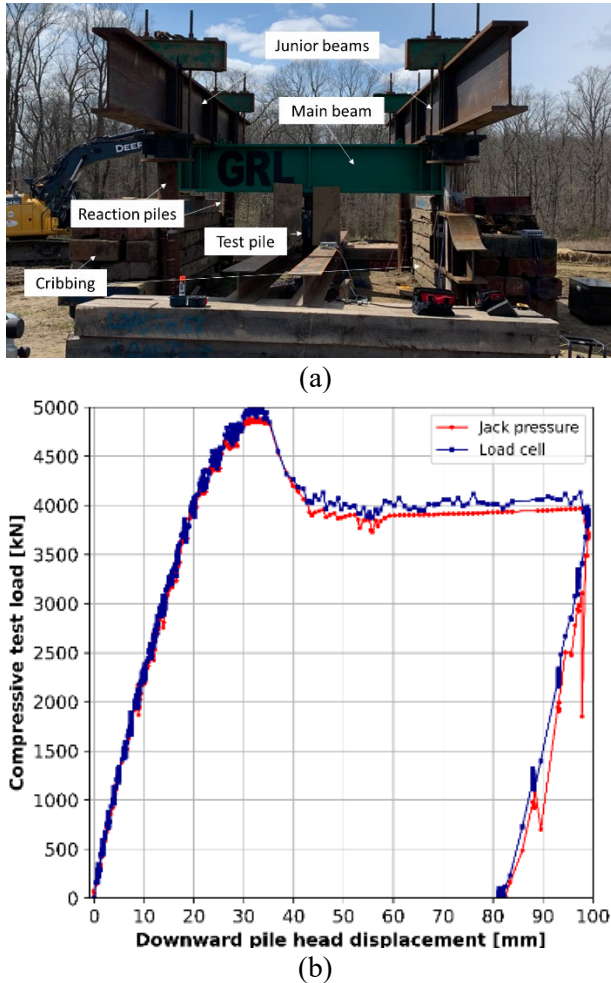


Figure 2. Compressive test on Pile S1: (a) equipment setup and (b) axial load-displacement response

Strain data from sister-bar strainmeters and FBGs were processed using the Incremental Rigidity method described by Komurka and Moghaddam (2020) and Komurka and Robertson (2020), to generate axial internal force profiles along the pile, with encouraging consistency despite some differences in strain measurements (Figure 3a). The load-transfer responses were evaluated using jack pressure to determine test loads, and internal forces were calculated from the recorded strains.

The t - z curves derived from strainmeter data across eight elevation intervals (referred to as “segments”) are shown in Figure 3b and generally align with CPT profiles, reflecting higher average unit shaft resistance in deeper, stronger layers: it is readily apparent that Segment 6, corresponding to a hard layer, showed the highest resistance. In contrast, shallow depths exhibited lower unit shaft resistance and a gradual reduction with increasing displacement, consistent with the expected response of degraded glauconite sand. Strain-softening in deeper layers was not observed, likely due to the analysis being limited to the peak global pile

response. Further analysis of strain data – not reported for brevity – indicated slow mobilization of toe resistance, with over 90% of the total measured resistance mobilized along the pile shaft. This may be a consequence of the Navesink hard layer, which inhibited axial load transfer to deeper soil locations.

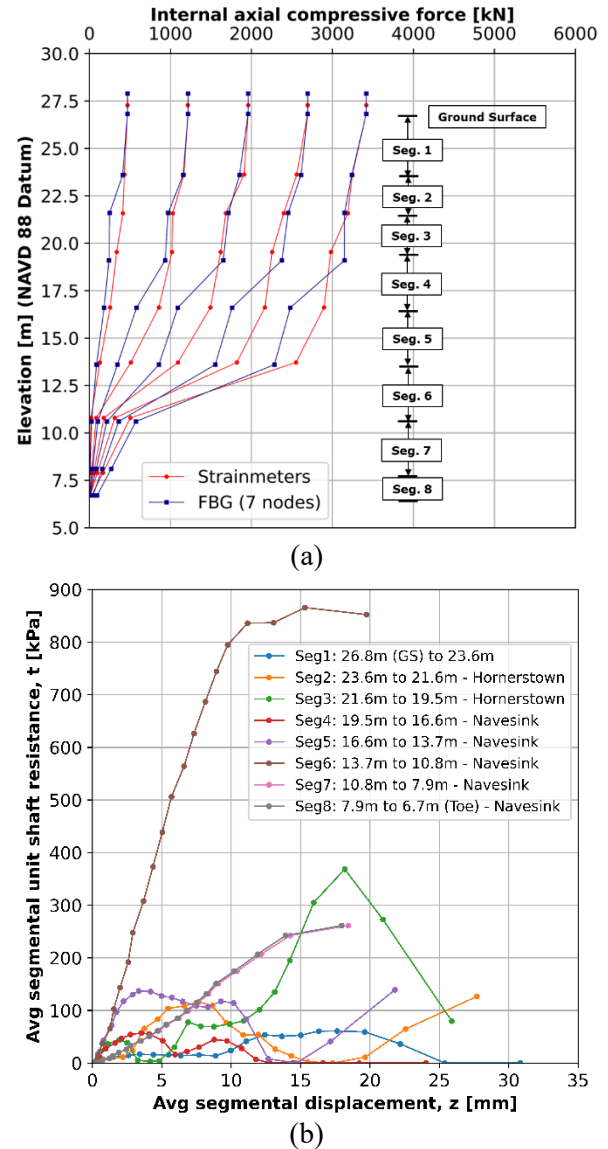


Figure 3. Pile S1: (a) Comparison between axial force profiles from strainmeters and FBGs; (b) experimental t - z curves inferred at different soil elevations

A static axial tensile load test was carried out on Pile S2, 27 days after its installation. The test results indicated that the pile achieved an ultimate capacity in the range of 2.7-2.9 MN, with the exact values dependent on whether load cell or jack pressure data was used (Figure 4).

In static load tests, loads indicated by a load cell and jack pressure often differ. The greater discrepancy in tensile test loads (cf. to Figure 2b) likely resulted from the pile being out of plumb and realigning during testing. Ultimate capacity was reached after an axial displacement of about 25 mm, slightly less than what was needed for Pile S1 to reach its compressive peak resistance. The test did not show any post-peak softening, in contrast to the response observed in Figure 2b. The stiffness during unloading was found to be similar to the initial stiffness at very low tensile loads (as low as ~10% of the ultimate capacity).

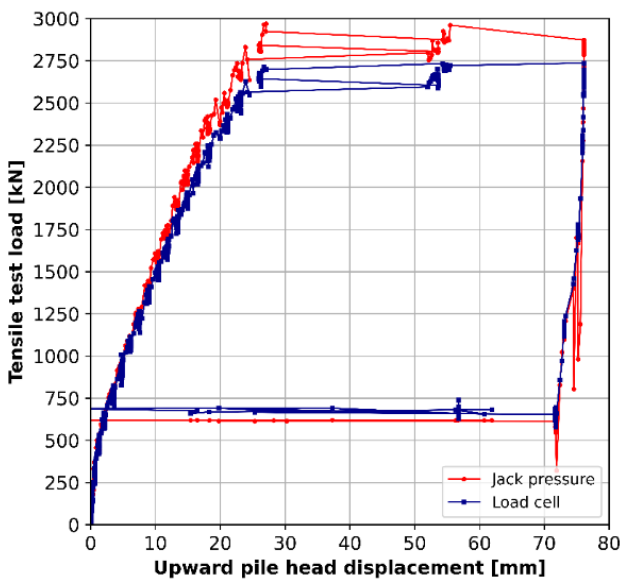


Figure 4. Pile S2: applied tensile axial load vs vertical pile head displacement

A comparative analysis of strain data for Piles S1 and S2 (details not reported for brevity) indicated generally compatible mobilized unit shaft resistance values for both piles – under compressive and tensile axial loading, respectively. Quantitative differences observed have been attributed to several factors, including: (i) Poisson effects of opposite sign during compressive versus tensile axial loading; (ii) uncertainties in estimating the toe resistance of S1 (not directly measured); and (iii) different embedment depths of Piles S1 and S2, leading to different relative positions within the two glauconite sand formations (Hornerstown and Navesink). Differences in setup are considered unlikely as installation and loading times were similar.

4 LATERAL LOADING

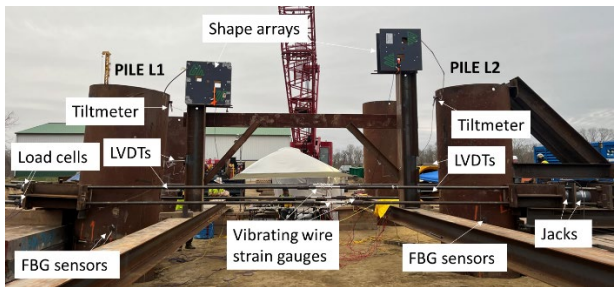
Lateral load tests were conducted on two pairs of piles, utilizing hydraulic jacks to pull the pile pairs

together, with each pile serving as a reaction for the opposing one (Figure 5a). The piles were equipped with instrumentation to measure applied loads, lateral displacement, rotation, and strain. Despite the symmetrical loading system, Pile L2 exhibited greater deflection than Pile L1, though the exact reason remains unclear and may involve several concurrent factors, such as the presence of an external shoe at the bottom of Pile L2 and the noticeable tilt of Pile L1 caused by installation operations (also visible in Figure 5a). Particularly, the 25 mm external shoe may have caused a temporary gap in the surrounding stiff Navesink formation during installation, and therefore a reduced lateral stiffness and greater deflection in L2 (~13 cm for L2 vs. ~10.5 cm for L1 at peak load – values measured at the lower LVDT, located ~1.5 m above ground surface, see Figure 5b). Both piles showed similar small-deflection stiffness up to about 500 kN. However, the overall response of the piles differed, with L1 demonstrating more flexible behavior and minimal toe kick. Derived bending moment and lateral deflection profiles are shown in Figure 5c-d (L2 only, as a representative simulation case), indicating the flexible behavior exhibited by relatively long piles in stiff glauconitic soil – therefore, not in line with the typical rigid, rotational response expected for modern, large-diameter monopiles.

Preliminary 1D numerical analyses were conducted using NGI's NGIPile program to assess field data from lateral load tests on piles. Piles were modeled as one-dimensional Timoshenko beams subjected to lateral loads, with soil reactions based on the PISA soil reaction base model for sandy soils. In the spirit of the PISA pile analysis method for sandy soils, the model used default calibrations for Dunkirk sand, combined with site-specific, pre-installation parameters such as small-strain shear modulus (G_{max}) and relative density (D_r). Different combinations of D_r values (100% and 70%) and D_r profiles (low-/best-/high-estimates, i.e., LE/BE/HE) were considered to assess the potential impact of uncertain soil properties. The results in Figure 6 show generally good agreement between measured and simulated data, particularly in the small-deflection regime. Differences in adopted G_{max} profile had limited impact, while underestimating D_r significantly reduced the accuracy of simulations. It appears as though glauconite crushing near the pile shaft has little influence on the global lateral load-deflection response, as this is influenced by the reaction forces of a substantially larger volume of soil around the pile.

5 CONCLUSIONS

This paper summarizes field testing and analysis of axial and lateral pile load tests at a glauconite sand site in New Jersey, providing a glimpse into the complexities of glauconite sand behavior at a highly characterized and instrumented test site. Recorded data and subsequent analysis (partly reported here) indicate that glauconite crushing influences pile response to varying degrees, depending on whether the response to axial loading (most affected) or lateral loading (less impacted) is considered. Future research will focus on addressing open questions about axial and lateral loading response of piles in glauconite sand. For axial loading, enhancing t - z and Q - z curves, comparing them with existing soil models, and integrating new laboratory results will improve industry capabilities for axial load-transfer analyses. For lateral loading, further development and validation of 3D numerical models and p - y curves are needed. Centrifuge testing may support these efforts to advance pile engineering for sites featuring glauconitic soil conditions.



AUTHOR CONTRIBUTION STATEMENT

F. Pisanò: Load test data curation, Formal analysis, Project management, Writing - Original draft. **Z. Westgate:** Conceptualization, Overall field data collection and curation, Formal analysis, Funding acquisition, Project management, Writing - Original draft. **A. Rahim:** Conceptualization, Load test data collection and curation, Formal analysis, Software, Project management, Writing - Review and Editing. **C. Maldonado:** Funding acquisition, Project management. **V. Komurka:** Conceptualization, Strain gauge data collection and curation, Formal analysis, Software, Writing - Review and Editing. **R. Beemer:** Conceptualization, Fiber optic data curation, Formal analysis, Software, Writing - Review and Editing. **E. Liedtke:** Conceptualization, Load test data collection. **Y. Perikelous:** Conceptualization, Load test data collection. **Additional Contributors:** Technical Steering, Writing - Reviewing and Editing.

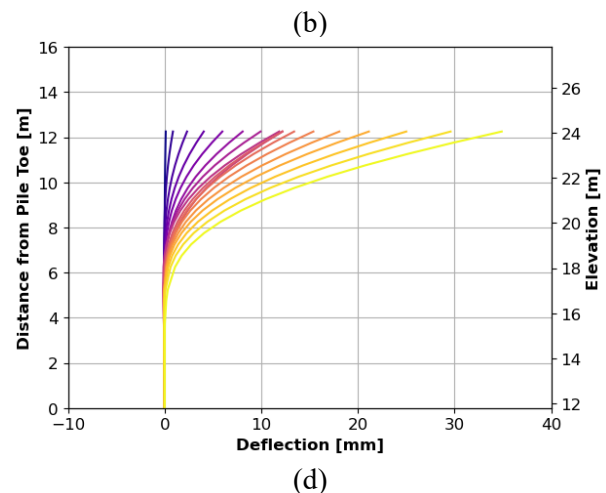
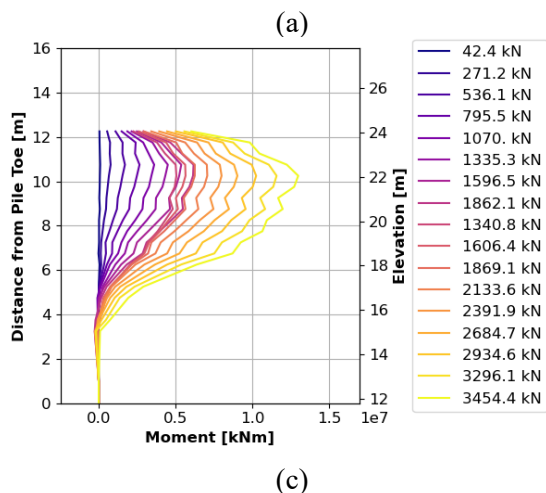
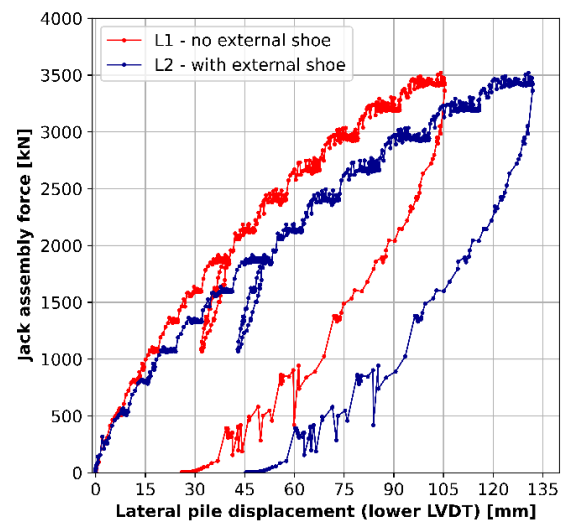


Figure 5. (a) Lateral load test setup and (b) measured lateral load-deflection responses for Piles L1 and L2; (c) bending moment and (d) lateral deflection profiles derived from FBG data for Pile L1 – legend indicates lateral load levels in kN

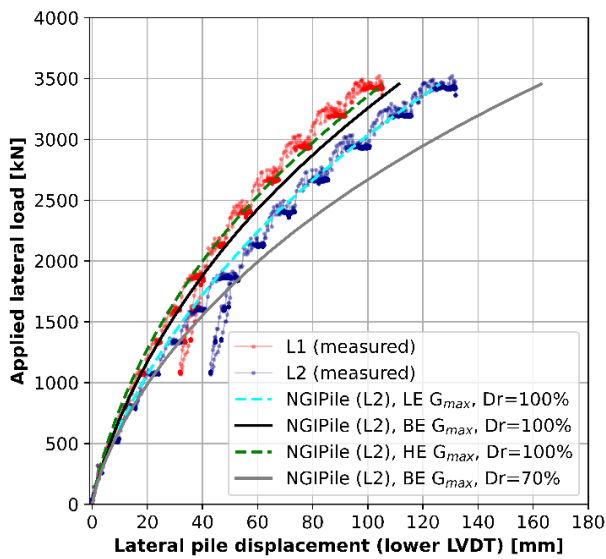


Figure 6. Comparison among measured lateral response Piles L1 and L2) and 1D numerical simulation results (L2 only, with different G_{max} and D_r assumed profiles)

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