

Field Observation of Shield Tunnelling and Its Transient Effects on A Loaded Pile

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ABSTRACT: A full-scale research study was conducted during the bored tunnelling beneath an existing building structure in the Klang Valley Mass Rapid Transit – Putrajaya Line in Kuala Lumpur, Malaysia. The primary objective of this study is to investigate tunnel-soil-pile interaction at different stages of tunnel excavation. The research involves a combination of full-scale field measurements and numerical analysis, contributing to a deeper understanding of the transient effects of shield tunnelling on existing loaded piles. An extensively instrumented experiment pile, equipped with vibrating wire strain gauges, inclinometer, and distributed fibre optic sensing using Brillouin Optical Time Domain Analysis (BOTDA), was installed, pre-loaded, and continuously monitored in real-time throughout the tunnel construction process. BOTDA is a novel technique for continuous strain measurement, provides inherent advantages over conventional point-based sensors in evaluating three-dimensional pile deformation such bending and axial displacements. The field measurement results indicate that the zone of influence due to tunnelling effects extended from a distance of 2 times the tunnel diameter behind the pile to 4 times the tunnel diameter ahead of the pile, with peak impacts observed at 1.5 times the tunnel diameter ahead of the pile. Continuous strain profiles from fibre optic sensors showed upward forces in the lower portion of the pile, contributing to tensile strains. The maximum tensile load was temporarily sustained before reverting to a compressive downdrag load as the tunnel boring machine (TBM) moved further away. The delayed response, likely occurring upon the dissipation of excess pore water pressure and diminishing tunnel face support pressure. The study elucidates distinct outward movement directions observed in the pile as the TBM approaches, followed by lateral deflection returning towards the tunnel upon TBM's passage. This validation strengthens understanding of pile behaviour during various tunnel excavation stages, supporting the negative face loss hypothesis.

KEYWORDS: Field Measurement, Tunnel-Soil-Pile Interaction, Distributed Fibre Optic Sensing, Transient Effects, and Pile Responses.

1. INTRODUCTION

A pile serves as a foundation element, typically a long and slender column, driven or installed into the ground to transfer structural loads into deeper and more competent soil or rock layers. The pile foundation achieves equilibrium when the structural integrity and load-bearing capacity of the pile are balanced against the applied loads, accounting for the gradual settlement and consolidation of the surrounding soil over time. The excavation of a tunnel disrupts the balance, leading to additional deformation and stress adjustments in both the surrounding soil mass and the pile foundation.

In the context of shield tunnelling, piles experience displacements in various directions depending on their location relative to the tunnel boring machine and the magnitudes of ground loss components such as face loss, shield loss, and tail loss (Loganathan, 2016). During tunnelling, it is presumed that the soil will “flow” both vertically and horizontally towards the center of the tunnel, resulting in a soil arching phenomenon (Anagnostou, 2012; Wu et al., 2022; He et al., 2023). The vertical soil movement above the tunnel springline is generally downward (due to volume loss and gravity). This downward movement may cause negative skin friction on the pile, inducing additional pile settlement if the pile is located near the tunnel. Conversely, the vertical soil movement below the tunnel springline is upward (due to stress relief), which may lead to pile heave. Piles with their bases located above or close to the tunnel axis level are potentially more adversely affected (Loganathan, 1999; Hong et al., 2015; Mohamad et al., 2022). This phenomenon is likely to induce additional axial forces and bending moments on the pile. Therefore, it is crucial to evaluate the capacity and serviceability of the piles in response to these tunnelling-induced transient effects.

The induced pile responses due to tunnelling have undergone extensive study over an extended period, involving field studies,

laboratory centrifuge tests, and various methodologies including empirical, analytical, and numerical approaches. While many case histories of tunnel-pile interaction have been reported in the literature, the examination of tunnelling-induced pile responses has typically been conducted indirectly through the monitoring of the structures and ground instrumentation. In essence, only a limited number of case studies involve in-pile instrumentation, offering the opportunity for a more in-depth investigation into pile response. Some notable reported field studies are summarized in Table 1.

Full-scale field research was conducted at the tunnelling site of Malaysia's Klang Valley Mass Rapid Transit – Putrajaya Line. The study involved installing, loading, and real-time monitoring of an experimental pile in close proximity to the route of twin bored tunnels excavated using Variable Density (multimode) Tunnel Boring Machines (TBM). The research builds on previous efforts and addresses gaps in knowledge by employing distributed fibre optic sensing technology based on stimulated Brillouin scattering, through the Brillouin Optical Time Domain Analysis (BOTDA). This novel technique allows for continuous measurement of strains, offering inherent advantages over the conventional point-based sensors (e.g., strain gauges) when assessing the interaction of forces between pile-tunnel structures and the soil (Soga et al., 2008). It is also effective in characterizing anomalies in cast in-situ concrete piles (Mohamad et al., 2016). The sensors integrated into the pile measure three-dimensional deformations, including bending and lateral displacements, providing more precise information about tunnel-soil-pile interaction and pile responses throughout the entire pile length during the transient effects of shield tunnelling. This cutting-edge technology has proven its effectiveness in monitoring the performance of various large civil and geotechnical structures. Some successful applications of this instrumentation to various geostructures include piles (Ohno et al., 2002; Klar et al., 2006), test

piles (Mohamad et al., 2009; Tee et al., 2016; Tee et al., 2017), retaining walls (Mohamad et al., 2011), as well as bridges and tunnels (Mohamad et al., 2010; Mohamad et al., 2012).

This paper presents the initial findings of the research, offering insights into crucial elements such as the experimental setup, the instrumentation regime implemented, the field observations collected throughout the monitoring period, and a detailed description of the

measurement results. The numerical modelling and comparison of its results with field measurements will be reported in a separate paper. The primary objective is to enhance current knowledge regarding the transient effects of shield tunnelling on a loaded pile, providing valuable insights for the broader scientific and tunnelling community.

Table 1 Summary of reported field studies on tunnelling-induced pile responses involving in-pile instrumentation

Citation	Project	Soil Type	Tunnel Type	Pile Type	In-pile Instrumentation
Mair (1993) Lee et al. (1994)	Angel Underground Station, London	London clay	Hand-dug tunnel	Bored pile (under-reamed)	Inclinometer
Teunissen & Hutteman (1998) Kaalberg et al. (2005)	North/South Line, Amsterdam	Very soft silty clay overlying sand layer	Shield tunnel	Timber pile, concrete pile	Pile top pressure sensor, precise levelling
Coutts & Wong (2000) Pang et al. (2005)	North-East Line C704, Singapore	Bukit Timah granitic residual soil	EPB shield tunnel	Bored pile	Vibrating wire strain gauge
Jacobsz et al. (2005) Selematas et al. (2005)	Channel Tunnel Rail Link C250, Essex, UK	Terrace gravel and London clay	EPB shield tunnel	Driven pile, bored pile	Vibrating wire strain gauge, base load cell, electrolevel inclinometer
Cham (2007)	Circle Line C852, Singapore	Bukit Timah granitic residual soil	EPB shield tunnel	Bored pile	Vibrating wire strain gauge
Mohamad et al. (2022)	Grand Paris Express Line 16, France	Fine clayey sand overlying limestone	EPB shield tunnel	Cast in-situ pile	Strain gauge, optical fibre cable

2. BACKGROUND OF THE STUDY

2.1 The research site

The Putrajaya Line, formerly known as Sungai Buloh-Serdang-Putrajaya Line, is the second of three planned mass rapid transit rail lines under the Klang Valley Mass Rapid Transit (KVMRT) project. The twin bored tunnels, each with a diameter of 6.35 m, were constructed using a closed shield-type tunnel boring machine and pre-cast steel fibre-reinforced concrete segments.

A designated full-scale research site was established at the Education Quarters situated within the central business district of Kuala Lumpur. The tunnels traversed through the Quarters (mid-rise apartments) from the northwest of Hospital Kuala Lumpur (HKL) Cross-over towards the southeast, ultimately reaching Raja Uda Station, as illustrated in Figure 1. The southbound tunnel specifically passed beneath a 5-storey building, posing challenges due to existing building foundations. To address this, the affected piles were removed, and underpinning work was carried out in preparation for TBM passage. This underpinning work involved installing micropiles capped under a reinforced concrete transfer structure to span across the tunnel. In anticipation of the complexities during tunnel construction, specific arrangements were made to install an additional experimental micropile. This strategic decision facilitated comprehensive in-pile instrumentation and real-time monitoring throughout the tunnelling process. This experimental pile played a critical role in providing detailed insights into the interactions and transient responses of the pile-tunnel system.

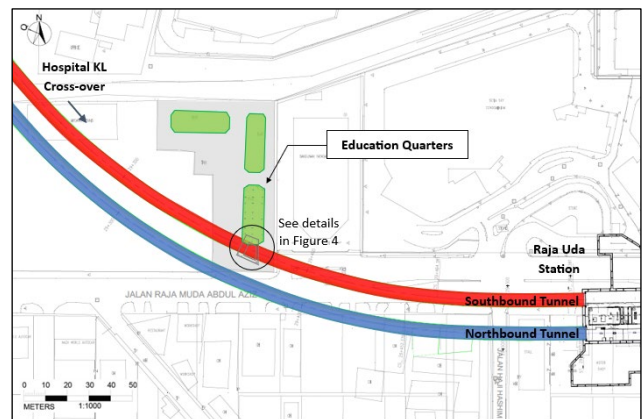


Figure 1 Location of the research site

2.2 General ground conditions

The site is underlain by the Kuala Lumpur Limestone Formation, as indicated by the geological map of Kuala Lumpur. The site's terrain is predominantly flat, with an average reduced level (RL) of 34 m above mean sea level. Site investigation boreholes have confirmed the anticipated regional geological setting, with alluvium overlying limestone bedrock. However, the bedrock levels at the site were predicted to be erratic, potentially containing cavities due to the inherent karstic features of limestone formation. The alluvium typically comprises interbedded layers of loose sand and soft clay/silt. Figure 2 illustrates the interpreted geological profile along the southbound tunnel alignment, while Figure 3 depicts the plot of Standard Penetration Test (SPT) values versus depth for overburden soils, revealing consistently low SPT values in the surficial alluvium above the tunnel springline. Moderately conservative design values have been selected for correlations of geotechnical characteristics. Basic soil parameters are summarised in Table 2. For a detailed derivation, readers can refer to Khoo et al. (2025).

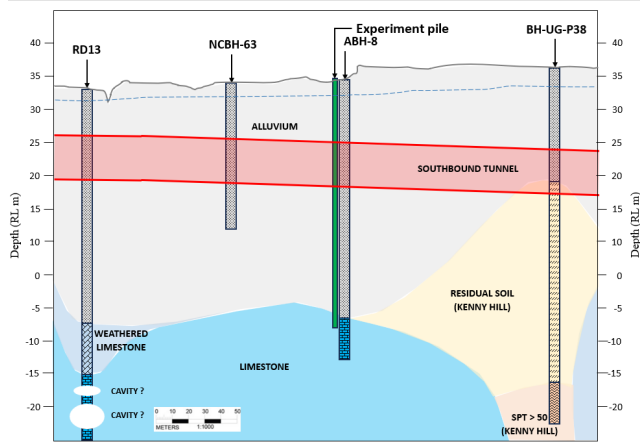


Figure 2 Interpreted geological profile along the southbound tunnel alignment

The general trend observed in the groundwater monitoring records indicates a gradual decline in groundwater levels from RL33.5 m at HKL Cross-over to RL30.5 m at Raja Uda Station. At the research site, groundwater was found at a depth of 4 m, with typical seasonal fluctuations of ± 0.5 m.

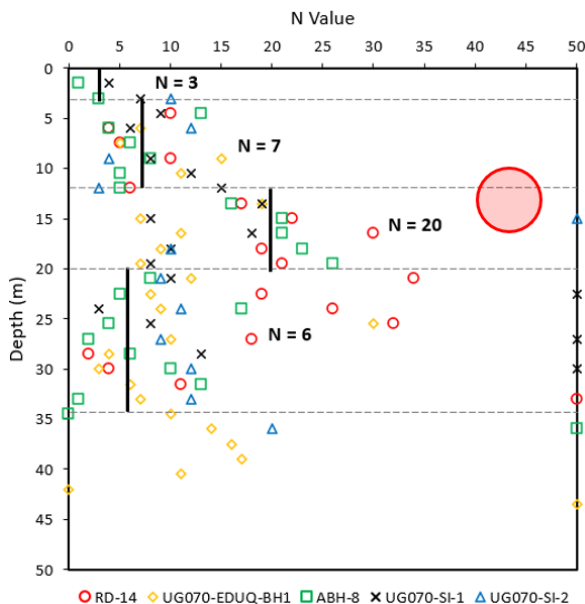


Figure 3 Selected SPT values for geotechnical characteristics correlations

Table 2 Summary of basic soil parameters

Parameter	Unit	Value
Bulk density	kN/m ³	15.2 – 21.0
Moisture content	%	15.0 – 41.0
Plasticity index	%	6.0 – 53.0
Effective cohesion	kN/m ²	1.0
Effective friction angle	°	30 (for SPT < 10) 32 (for SPT ≥ 10)
Modulus of elasticity (undrained)	MN/m ²	2.5N (for SPT < 10) 2.0N (for SPT ≥ 10)

Note: N = SPT value

2.3 Underpinning scheme and pile removal

The underpinning work involved seven 330 mm diameter micropiles connected by a 1 m-thick reinforced concrete transfer structure spanning the tunnel. This structure was designed to safely support the column loads before the removal of the affected piles. Figure 4 provides a plan view and cross-section of the tunnel in relation to the

existing building foundation, newly installed underpinning piles and the experiment pile.

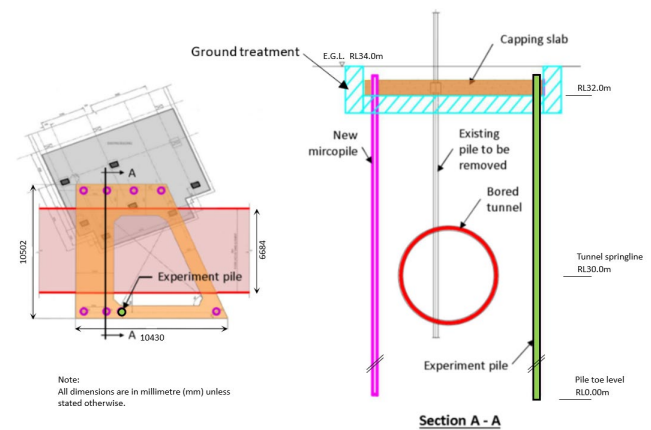


Figure 4 Plan view and cross-section of the underpinning works

To facilitate excavation for the construction of the transfer structure, ground treatment in the form of jet grout columns ($q_u = 600$ kN/m²) was employed as a temporary retaining structure, along with 1 m-thick base grout forming the soffit. Tan et al. (2019) detailed the methodology used to remove the existing 300-mm-square reinforced concrete piles identified as in conflict with the tunnel.

3. THE EXPERIMENT PILE AND INSTRUMENTATION

An experiment pile, with a diameter of 300 mm and located at 1.5 m away from the extrados of the southbound tunnel, is reinforced with API (American Petroleum Institute) pipe measuring 177.8 mm in outer diameter and 10.36 mm in wall thickness. Installed to a depth of 1.5 m into the limestone bedrock at 32.3 m below ground level, the micropile (grade G30) is designed with a working load of 600 kN. Key details of the experiment pile are summarized in Table 3.

Table 3 Details of the experiment pile

Parameter	Unit	Value
Pile diameter	mm	300
Pile working load	kN	600
Pile reinforcement	-	API pipe 177.8 mm O.D. with 10.36 mm wall thickness
Cement grout grade	MN/m ²	30
Water cement ratio	-	0.45 : 1.00
Rock socket length	m	1.5
Total pile length	m	33.8
Date of installation	-	16 March 2018

The pile head protruded above the transfer structure within a pre-installed 400-mm-diameter, 6-mm-thick steel sleeve, as shown in Figure 5. A temporary steel casing was left in place until the soffit of the ground treatment layer (3 m below ground level) to protect the pile head section and ensure the cabling survived the construction process. The reaction anchorage method was employed to apply the load to the experimental pile. A load cell measured the amount of load transferred to the pile, while linear voltage displacement transducers installed on an independent reference frame measured pile top settlement, along with precise optical levelling.

During the preloading stage, jack loads were incrementally increased up to 600 kN in 10 steps, with each step held for 10 minutes, and strain distribution along the pile was measured. The working load was maintained throughout the tunnel advance monitoring period. This study assumes that the working load applied to the pile remains constant and does not change due to tunnel excavation, which may be considered acceptable if the superstructure is flexible and statically determinate.

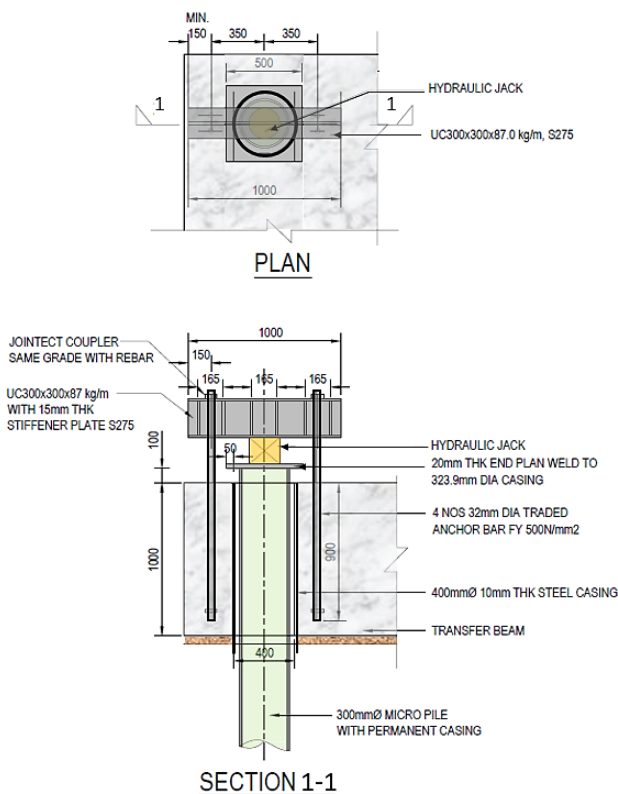


Figure 5 Pre-loading jack configuration

The in-pile instrumentation was meticulously designed and arranged to offer real-time data on the tunnelling-induced transient pile responses. This includes monitoring pile head vertical displacement, pile shaft horizontal deformation, pile axial load distribution, and internal forces within the pile. The instrumentation setup comprised both conventional vibrating wire strain gauges and an inclinometer, alongside innovative fibre optic distributed sensors. The integration of both conventional and advanced sensing technologies enhances the reliability and accuracy of the measurements, facilitating a thorough evaluation of the pile's response to the tunnelling-induced transient effects. The details of the in-pile instrumentation and arrangement are illustrated in Figure 6.

Six levels of vibrating wire strain gauges were strategically affixed to the pile reinforcement (API pipe). These gauges were positioned at 3-meter intervals, commencing from a depth of 4 m below the ground surface. Each level corresponds to significant measuring points: Level 1 – 6 m above tunnel crown, Level 2 – 3 m above tunnel crown, Level 3 – tunnel crown, Level 4 – tunnel springline, Level 5 – tunnel invert, and Level 6 – 3 m below tunnel invert. At every level, the strain gauges were meticulously arranged on four sides (two pairs per level), with each pair aligned parallel and perpendicular to the tunnel orientation. These strain gauges, spot-welded on a 10 mm steel band, were then securely fastened to the designated levels on the API pipe. Unfortunately, some sensors did not survive the installation process.

The fibre optic sensing cable is composed of a single-core single-mode optical fibre reinforced with six strands of steel wires and a polyethylene cable jacket. This cable was affixed along the two opposite sides of the API pipe, with a loose or coiled section at the pile tip. It was then extended along each side of the pipe as it was lowered into the borehole, creating two loops (A1-A2 and B1-B2). These loops were spaced equidistantly in directions perpendicular (axis x-x) and parallel (axis y-y) to the tunnel alignment. Additionally, a pair of temperature-sensing cables (T1-T2) were installed alongside the A1-A2 cable to measure temperature changes and compensate for any potential differential temperature effects on strain readings.

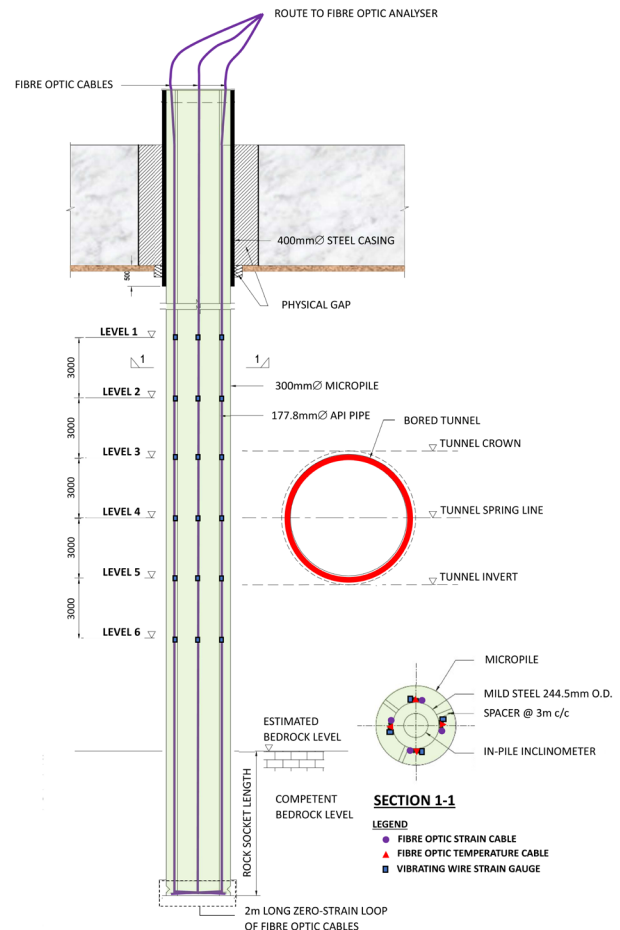


Figure 6 In-pile instrumentation and arrangement

Throughout the monitoring period, temperature variations and fluctuations within the pile were deemed insignificant (Khoo, 2024). Each optical sensing cable used in the field measurement underwent laboratory calibration, with a strain coefficient of $20 \mu\text{e}/\text{MHz}$. All fibre optic cables were pretensioned to maintain a straight line along the main axes x-x and y-y, enabling the identification of sensing section positions when analysing the data traces.

Additionally, an inclinometer casing was strategically pre-installed inside the API pipe during the grouting process, providing redundancy for pile deflection measurements and ensuring comprehensive data collection on the pile's behaviour during the tunnelling process. Figure 7 illustrates the actual setup at the site. The baseline measurement, used for interpretation, was taken one month before the TBM reached the target to eliminate any strain induced by the installation process.

In addition to the in-pile instrumentation, settlement markers, inclinometers, and piezometers were installed in the ground adjacent to the experiment pile. The building that the tunnel undercrossed was also closely monitored. This comprehensive instrumentation regime allowed for a thorough investigation of tunnelling-induced effects and tunnel-soil-pile interaction. Figure 8 illustrates the ground and structure instrumentation surrounding the experiment pile. For the purpose of this paper, only relevant instrumentation data will be presented.



Figure 7 Setup at the site

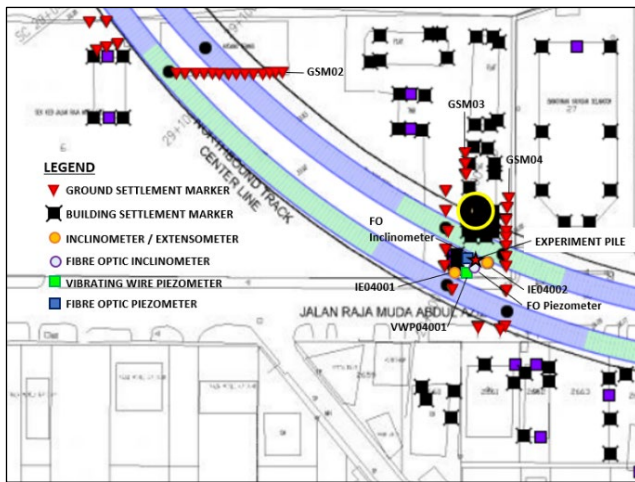


Figure 8 Ground and structure instrumentation

4. TBM ADVANCE AND DRIVING DATA

The installation of the instrumented experiment pile was completed in March 2018, followed by pile removal work in advance of the passage of the TBMs. Construction activities were carefully coordinated to ensure that the underpinning and pile removal works did not interfere with tunnelling activities or affect the permanent tunnel lining. Consolidation due to piling and ground treatment works was presumed to have been completed prior to TBM advance, given the high permeability of the alluvium sandy soil.

A variable density tunnel boring machine was chosen for excavating the tunnels from HKL Cross-over to Raja Uda Station. The key technical data for this machine is summarised in Table 4. Further details can be referred to Schaub & Duhme (2014).

The TBMs in this tunnelling section were operated in the EPB mode. This study specifically focused on the southbound TBM advance for a distance between approximately $y = -10D$ to $10D$ from the experiment pile, where y is the distance between the tunnel face and the experiment pile, normalised by the tunnel diameter (D). Positive values of y indicate distance where the TBM passed beyond the pile.

Figure 9 illustrates the time-location of the TBM, and Table 5 provides the recorded tunnel mining and ring build dates for the respective locations. In summary, this stretch of the tunnel was completed between 3 May 2019 and 16 May 2019. It is important to note that the TBM stopped for planned cutterhead intervention on 6 and 7 May 2019, which was carried out in a grout block, as indicated in Figure 9. For clarity in subsequent charts, the solid lines represent the time at which the tunnel boring machine's face first reached a particular distance, with a solid, bold line indicating when the tunnel

face reached the experiment pile location. The excavation at the cutting face, the tail void grouting, and the erection of a new lining ring during standstill are considered simultaneously for the purpose of this paper.

Table 4 Key technical data of the tunnel boring machine

Machine type	Variable Density TBM
Length TBM + back-up	ca. 135 m
Weight TBM + back-up	ca. 876 t
Diameter front shield	6620 mm
Length front shield	3755 mm
Diameter centre shield	6610 mm
Length centre shield	3645 mm
Diameter tail skin	6600 mm
Length tail skin	3785 mm
Correction curve radius (min.)	150 m
Working pressure (at axis)	5 bar
Cutterhead power	1280 kW
Torque	4329 kNm

Tunnelling modes:

Mode 1: EPB closed

Mode 2: EPB closed with additional bentonite support

Mode 3: Mixshield mode with LDSM

Mode 4: Mixshield mode with HDSM

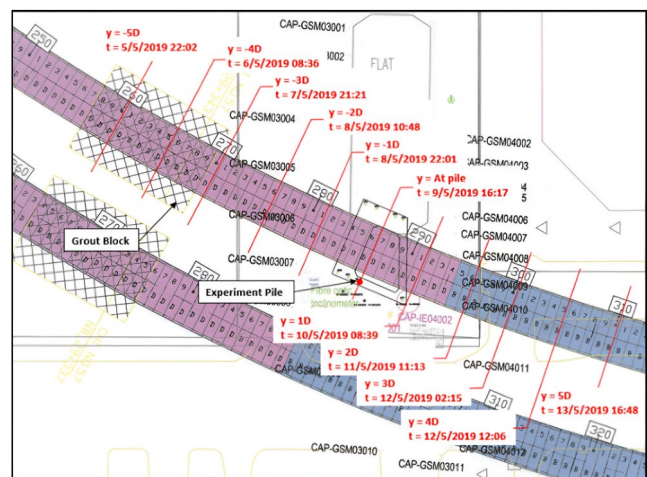


Figure 9 Time-location of the southbound TBM in relation to the experiment pile

Table 5 Record of tunnel mining and ring build dates

TBM Location, y	Ring No.	Mining Date	Ring Build Date
-10D	238	3 May 2019	4 May 2019
-5D	260	5 May 2019	6 May 2019
-4D	265	6 May 2019	8 May 2019
-3D	270	7 May 2019	8 May 2019
-2D	276	8 May 2019	9 May 2019
-1D	281	8 May 2019	9 May 2019
At pile	286	9 May 2019	10 May 2019
1D	291	10 May 2019	11 May 2019
2D	296	11 May 2019	12 May 2019
3D	300	12 May 2019	12 May 2019
4D	305	12 May 2019	13 May 2019
5D	310	13 May 2019	14 May 2019
9D	329	13 May 2019	16 May 2019

During TBM progress, a continuous range of face support pressure was provided. Figure 10 illustrates the allocation of the six earth pressure sensors over the cutter head in the excavation chamber. Figures 11(a) and 11(b) show the recorded individual point pressures and averaged pressure, respectively. It is evident that a higher face pressure was applied ahead of the experiment pile. The face pressure gradually increased from 2 bar (at a distance of $y = -3D$) to a maximum of 2.6 bar as the TBM cutterhead reached the experiment pile location, eventually decreasing to the normal face pressure at a distance of $1D$ away.

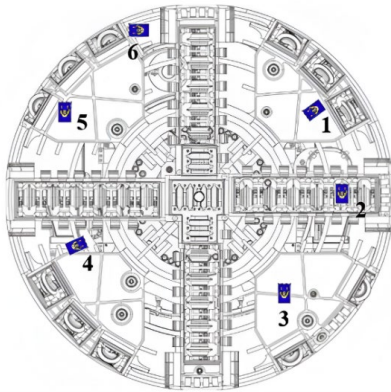


Figure 10 Allocation of TBM face support pressure sensors

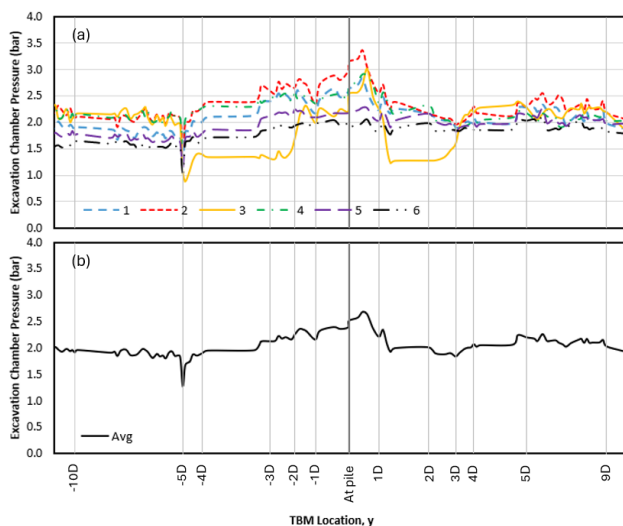


Figure 11 Recorded excavation chamber pressures (a) individual point pressure, (b) averaged pressure

As part of the TBM driving data, the normal range of advance speed was 20 – 50 mm/min, with an average of 34.5 mm/min for this length of bored section. Advance thrust forces and advance group pressures fluctuated between approximately 10 – 17 MN/m² and 50 – 150 bar, respectively. The grouting pressure at the grout ports remained relatively constant throughout the tunnel drive, with averaged pressures around 2-4 bar.

5. MEASUREMENT RESULTS

The analysis of all signal responses from the fibre optic sensors was conducted and compared with readings obtained from conventional instrumentation methods such as vibrating wire strain gauges and an inclinometer.

5.1 Pile head settlement

Figure 12 illustrates the overall load settlement curve of the experimental pile subjected to preloading and influence from tunnel advance. Pile head settlement was measured every 10 minutes during the preloading stage and at hourly intervals thereafter up to the maximum load during tunnel advance. The maximum load of 600 kN

caused a pile head settlement of 1.3 mm before tunnelling effects. The pile experienced a total settlement (due to working load and tunnelling) of 1.8 mm. After completing the tunnelling process, the pile was fully unloaded, and a residual settlement of less than 0.2 mm was recorded, indicating that the pile is generally still responding in a reasonably elastic manner.

Figure 13 shows the settlement measured at the pile head together with the applied jacking loads corresponding to the southbound tunnel advance period. Basically, pile head settlement increases when the tunnel enters a zone of approximately $y = -2D$ and beyond. This is due to the pile being subjected to larger shear strain caused by tunnelling stress release. Attempts to re-establish the target load resulted in negligible temporary compression near the pile head, which was quickly offset by the mobilization of additional shaft resistance in the lower pile, rendering this effect negligible. From this study, the zone of influence due to tunnelling effects can be considered at $y = -2D$ to $y = 4D$, slightly more than the coverage reported by Lee (2012) and Mroueh & Shahrour (2002). The maximum rate of increment of pile head settlement is observed when the tunnel has passed the experiment pile at $y \geq 1D$ and reaches its peak at $y = 1.5D$ (where the tail shield is at the pile location). This indicates that the incremental settlement was amplified during the passage of the shield and tail void grouting.

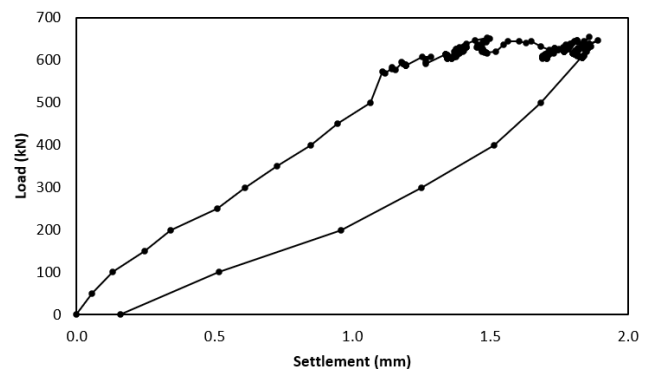


Figure 12 Load settlement behaviour of the experiment pile

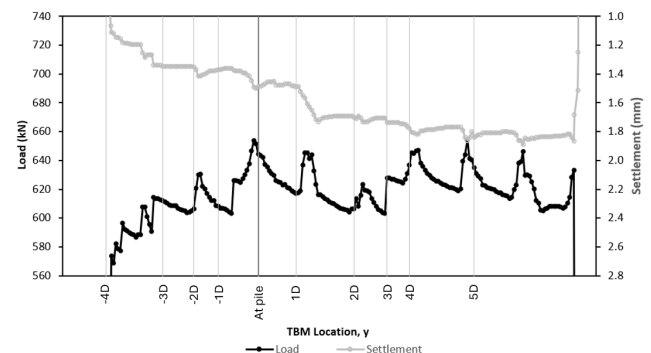


Figure 13 Incremental pile head settlement versus applied jacking load

5.2 Pile axial force

Figure 14 provides an overview of the measured (averaged) axial strain profiles using fibre optic sensing, with reference to borehole ABH-08 located at the experiment pile location. The strain profiles are presented in terms of the date of measurement (daily average) during the TBM advance.

Reiterating the earlier discussion, it is worth noting that a temporary steel casing was left in place until 3 m below ground level and therefore the cross-sectional area of the pile is expected to be consistent within this region. However, the observation of localised strain spikes raises uncertainties regarding their specific causes. One potential explanation could be the presence of laitance, which might not have been fully flushed out before reducing the standard casing length to 3 m. Additionally, localised higher strains are noted at

depths ranging from approximately 3 to 5 m below the pile top level. This depth coincides with the area just below the temporary casing, where sandy gravel materials were identified in borehole ABH-08 (refer to Figure 14). Most probably, some soil from this granular layer may have collapsed into the shaft and contaminated the grout during the withdrawal process of the temporary casing. As a consequence, the integrity of the grout was compromised, reducing its Young's modulus and contributing to relatively higher strain values. Mohamad et al. (2016) have previously demonstrated that distributed strain measurements are effective in identifying typical defects in bored cast in-situ piles.

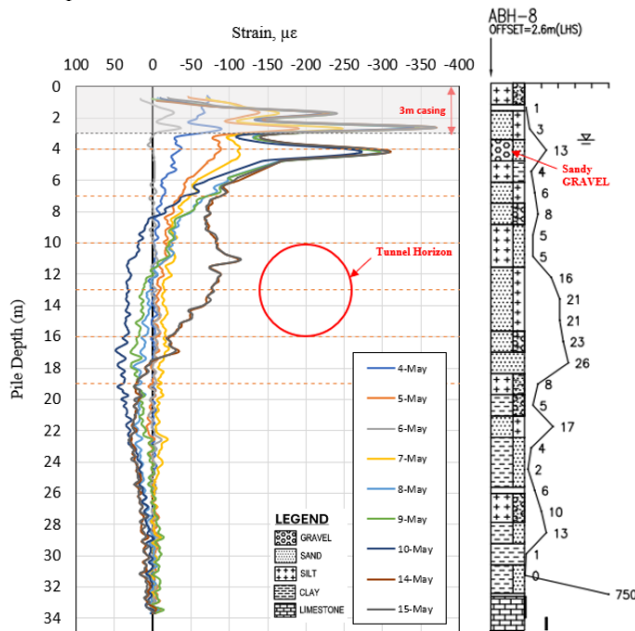


Figure 14 Axial strain profiles measured by fibre optic sensing

Figure 15 illustrates the transient behaviour of axial load distribution in the experiment pile, comparing between fibre optic sensor (FOS) and vibrating wire strain gauge (VWSG) data for six measurement dates corresponding to critical TBM advance period; (i) Pre-loading stage (-3D / 7 May 2019), (ii) TBM approaches (-2D / 8 May 2019), (iii) Arrival of TBM face (At pile / 9 May 2019), (iv) Passing of TBM shield (1D / 10 May 2019), (v) TBM passed beyond the experiment pile (5D / 13 May 2019), and (vi) TBM passed far beyond (9D / 15 May 2019). Compressive load is denoted as positive, while tension load is negative. Note that fibre optic measurement data is unavailable from 11 to 13 May 2019 due to unforeseen circumstances. Excellent agreement between the FOS and VWSG measurements on all measurement dates.

The axial strains and loads in the experimental pile responded to the TBM's effect after the preloading stage on 7 May 2019. When the TBM reached $y = -2D$ on 8 May 2019, field measurements indicated the initiation of tensile load detection at depths below the tunnel springline (between 13 m and 25 m). The upper part of the pile experienced higher compressive loads, suggesting possible downdrag induced by settling subsoil. On-site, temporary pile compressions were counteracted by re-jacking and the mobilisation of additional shaft resistance in the lower pile, mitigating the manifestation of negative skin friction at this stage. Furthermore, a much higher tunnel face support pressure was applied, resulting in a portion of the pile experiencing tensile stress. Another plausible explanation for this behaviour is that the lower portion of the soil tends to force the pile upward due to ground heaving and stress release caused by tunnel excavation.

The tensile load profile extended above the tunnel springline, reaching its peak once the TBM (tunnel face) passed beyond the experiment pile at $y = 1D$ on 10 May 2019. The highest tensile load measured from fibre optic data was 120 kN, which occurred below the tunnel invert level, leading to a reduction in compressive load

along the upper portion of the pile, as observed in the larger pile head settlement in Figure 13.

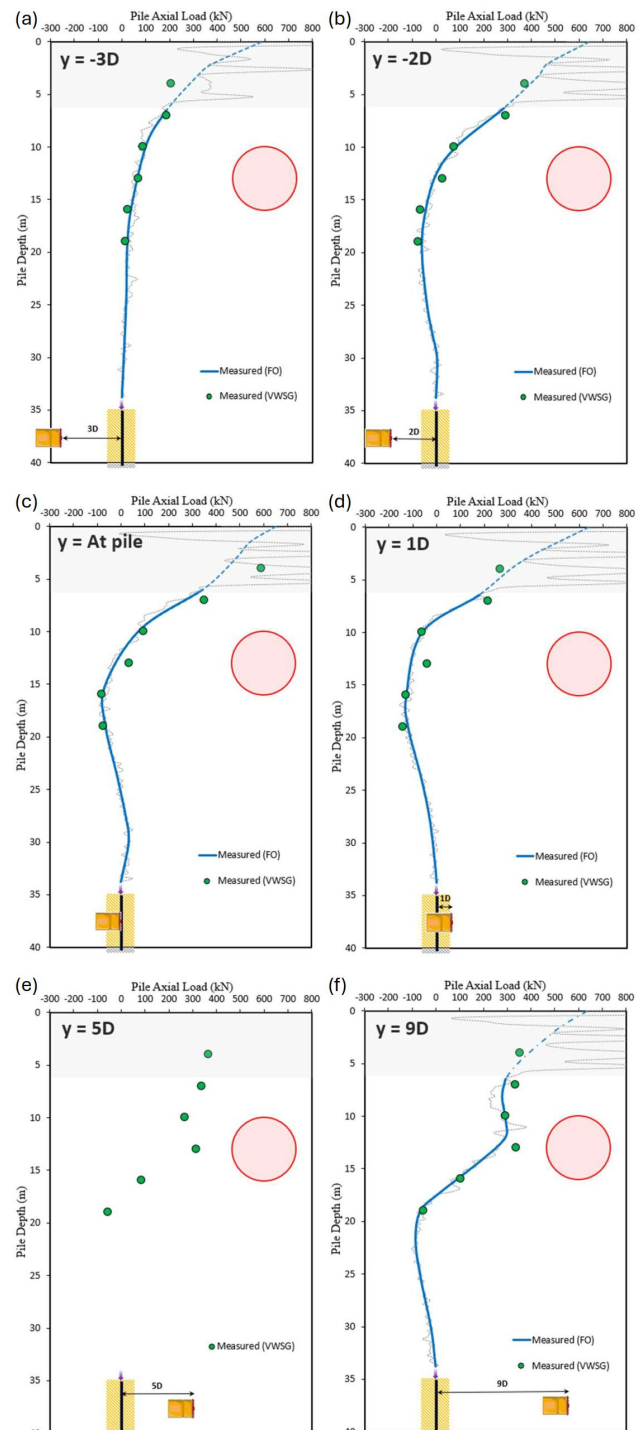


Figure 15 Transient axial load distribution of the experiment pile at (a) pre-loading stage (-3D / 7 May 2019), (b) TBM approaches (-2D / 8 May 2019), (c) arrival of TBM face (At pile / 9 May 2019), (d) passing of TBM shield (1D / 10 May 2019), (e) TBM passed beyond the experiment pile (5D / 13 May 2019), and (f) TBM passed far beyond (9D / 15 May 2019)

As the pile settled, shaft resistance along the upper portion was redeveloped, and negative skin friction from ground settlements imposed substantial downdrag load into the pile at the tunnel horizon. Conversely, when the TBM passed further away ($y > 5D$) ahead of the pile and tunnel linings were erected, the tensile stress within the tunnel horizon reverted to compressive stress as confirmed by field measurements. This indicates a delayed response of downdrag load upon the dissipation of excess pore water pressure and diminishing

tunnel face support pressure (see Figure 11). Figure 16 illustrates the changes in pore water pressure measured by both vibrating wire piezometers and Fibre Bragg Grating (FBG) piezometers during the tunnel advancement, where the rapid dissipation of excess pore pressures immediately after the TBM head passes through resulted in transient negative excess pore pressures, potentially leading to the delayed occurrence of downdrag load when the pore water pressures stabilised at a distance away (Khoo, 2025).

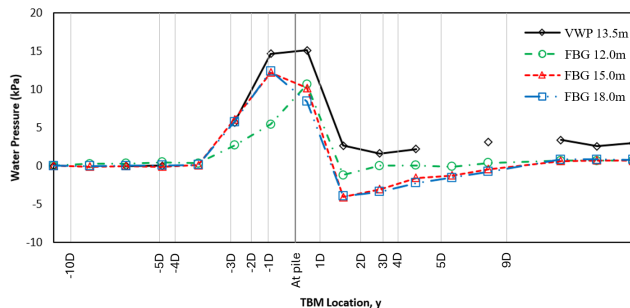


Figure 16 Changes in pore water pressures

Perhaps, the axial compression load in the pile increased without additional top load, providing clear evidence of downdrag (negative skin friction) caused by tunnelling-induced ground settlement. The measured tensile stress reached a residual value, with the lower part of the pile, i.e., from 2 m below tunnel invert level, remaining in a tension state. All in all, the distributed fibre optic data is better at describing the complete behaviour of the pile compared to the discrete data of VWSG (Hisham et al., 2023). In this case, the experimental pile had undergone various tension and compressive loadings due to the interacting tunnelling effects.

5.3 Pile bending moment

Figures 17(a) and 17(b) present envelope profiles of pile bending moment derived from fibre optic strain measurements for Axis A-A and Axis B-B, respectively. The measurements indicate that the pile only encountered a modest bending moment of 15 kNm, primarily above the tunnel crown. Importantly, the highest value remains well within the ultimate bending moment capacity of the pile.

This observation aligns with Pang et al. (2005), where the tunnelling-induced bending moment in the pile is considerably small and only up to the cracked moment. Having said that, proper control of volume loss during tunnelling will be necessary to limit the induced bending moment.

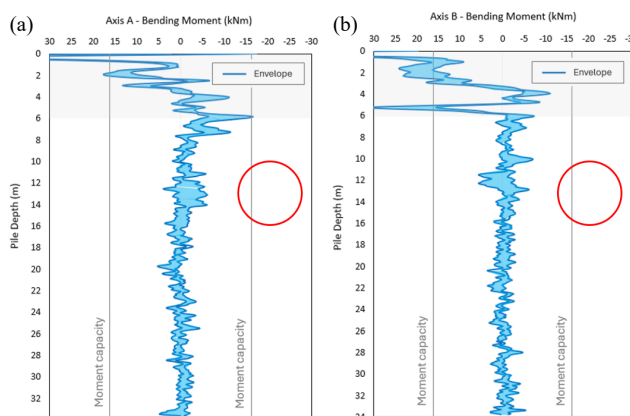


Figure 17 Measured pile bending moments of the experiment pile

5.4 Pile lateral deflection

Figure 18 shows the lateral deflection of the experimental pile, derived from fibre optic (FO) data and inclinometer IN04003 readings. Unfortunately, the inclinometer data only spans from 13 to 15 May 2019, with a notable absence of readings from earlier dates

due to unforeseen circumstances. To derive meaningful insights, the inclinometer data were differentiated once to obtain curvature and integrated once to obtain lateral displacement. For this presentation, positive values along Axis A denote movement away from the tunnel in the transverse direction, while positive values along Axis B indicate movement away from the tunnel face in the longitudinal direction.

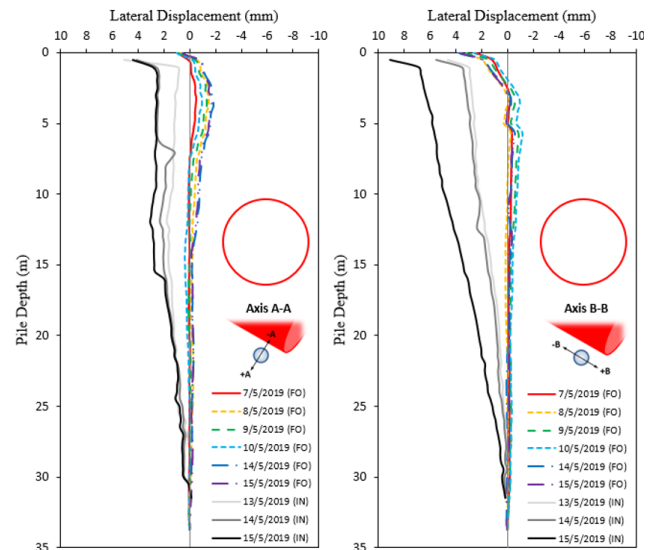


Figure 18 Interpreted pile lateral deflections at different stages of tunnel advance

Despite these efforts, the inclinometer readings did not yield significant interpretations. This inadequacy is likely due to improper preparation of the inclinometer socketing toe, which terminated within the grouted API pipe toe level. Furthermore, the inclinometer's spatial resolution of 0.5 m limits its capacity to capture fine local movements and perform precise curvature measurements. In contrast, FO-derived lateral displacement requires careful determination of two boundary conditions (constants A and B), whereas the inclinometer needs only constant B.

Figure 19 provides a closer view of the time-based pile movement path in the horizontal plane at the six measurement depths. The FO data indicates that the pile was generally being pushed outward both in transverse and longitudinal directions as the TBM approached the experimental pile. Once the TBM head had passed the pile, the lateral deflection progressively shifted toward the tunnel as the distance increased. Concurrently, the maximum lateral deflection shifts to the pile head, where the lateral soil movement is largest in the transverse settlement trough.

These observations align with the hypothesis of pile movement path proposed by Loganathan (2016) for a negative face loss scenario. As the TBM face pressure exceeds the mobilised earth pressure at the face, the ground, and consequently the pile, is pushed away from the TBM face, inducing heave at the surface and sub-surface ground movement away from the TBM face. This is evidenced by the measured ground-surface movements depicted in Figure 20. Additionally, the measurement of building settlement, as shown in Figure 21, further validates these observations. Refer to Figure 8 for the locations of the instrumentation.

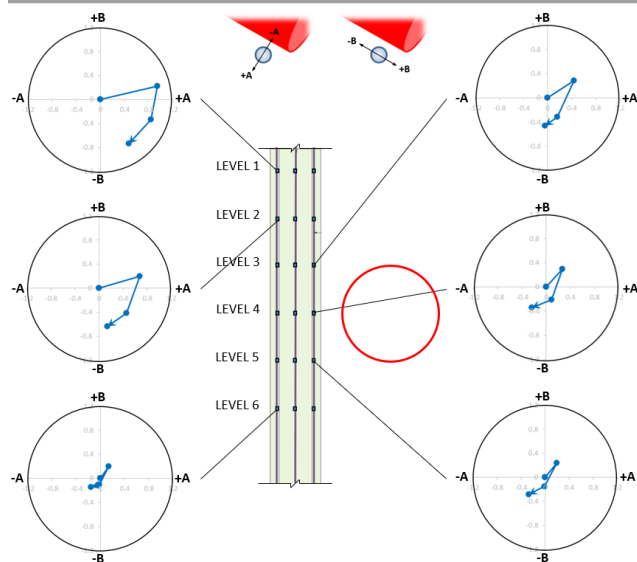


Figure 19 Time-based pile lateral deflections at different depths

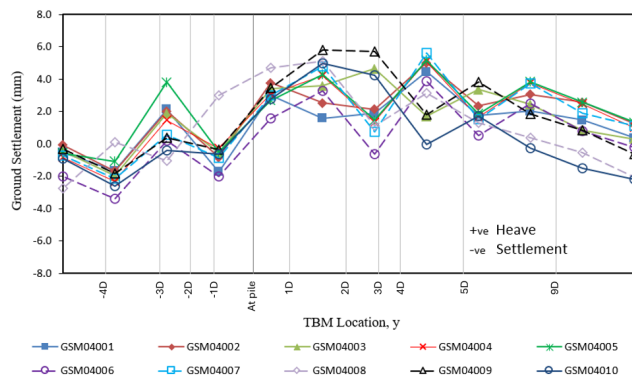


Figure 20 Measured ground surface movements

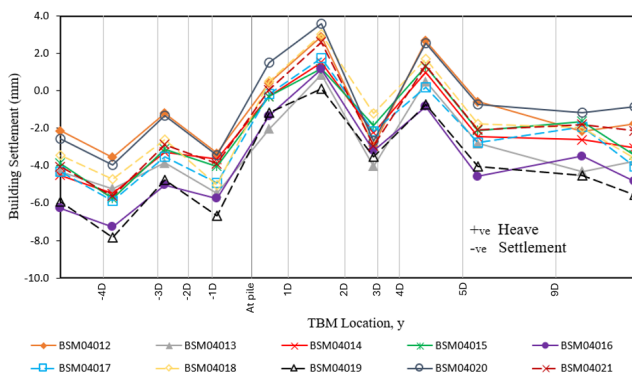


Figure 21 Measured building settlements

6. CONCLUSIONS

This paper provides comprehensive field observations focusing on the transient effects of pile responses due to tunnelling in a full-scale research study. The experimental pile, strategically located within 0.5D from the tunnel extrados, was designed as a rock-socketed pile subjected to working load and meticulously monitored throughout the entire tunnel construction process using in-pile instrumentation. The measurement of tunnelling-induced pile transient responses was successfully demonstrated using distributed fibre optic sensors via BOTDA and cross-validated with vibrating wire strain gauges, showing excellent agreement between continuous strain measurements from fibre optic and point-based strain gauges.

The field measurement results indicate that the zone of influence due to tunnelling effects extended from a distance, $y = -2D$ to $y = 4D$, with the peak observed at $y = 1.5D$. Consistent with findings from other researchers, tunnelling activity induces downdrag load on the adjacent pile, typically increasing until negative skin friction is fully

mobilised, with the maximum load occurring around the tunnel horizon. In this case study, tension force developed up to a maximum of 120 kN below the tunnel invert level in the lower part of the pile, propagating upward as the TBM passed through. This tension force is likely due to increased face support pressure and ground heave, resulting from stress release caused by tunnel excavation. However, the tension force eventually subsided, and compressive stress developed due to negative skin friction. This indicates a delayed response of downdrag load due to the dissipation of excess pore water pressure and the diminishing tunnel face support pressure. Importantly, both axial force and bending moment remain well within the pile capacities.

Based on the recorded TBM driving data and instrumentation monitoring, the findings strongly support the hypothesis of pile movement paths in a negative face-loss scenario. This research study highlights the significance of investigating transient pile responses to advancing shield tunnel. Overall, these findings collectively advance the understanding of tunnelling-induced effects on piles, paving the way for safer and more efficient underground construction practices.

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