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Axial load testing of FBG-instrumented piles in dense sand test site in Kilmuckridge, Ireland

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ABSTRACT: Significant advances in strain instrumentation has enabled researchers to determine pile behaviour during installation, equalisation and loading with increased accuracy. Optical fibre Bragg grating (FBG) strain sensors have been shown to provide accurate, stable measurements of strain for steel piles during driving without issue. This paper presents the results of a 406 mm diameter open-ended steel pile which was driven to a depth of 5.75 m in dense sand at Trinity College Dublin's geotechnical research test bed in Kilmuckridge, southeast Ireland. The pile was instrumented with FBGs to provide insights into pile behaviour during installation and subsequent static tension testing. The results showed that thermal effects during driving had a significant effect on the measured installation strains. Reasonable estimates of residual loads induced by the installation process were obtained through a back-analysis procedure using strain profiles at the end of the tension test.

1 INTRODUCTION

The offshore wind energy sector has experienced significant growth worldwide in recent years, particularly in Europe and Asia. To exploit the benefits of stronger, consistent wind resources, such expansion is accompanied by increasing water depths in which foundation systems must be sufficiently robust to withstand large static and dynamic environmental loads.

Significant insights into the behaviour of pile foundations for offshore settings have been reported by Lehane (1992), Chow (1997), Jardine et al. (2006), Gavin and Igoe (2019), Igoe and Gavin (2021) and Buckley et al. (2017, 2023). In these studies, the test piles were instrumented with strain sensors for determining the axial force during installation, equalisation and static loading using the following equation:

$$F = EA\varepsilon \quad (1)$$

where F is the axial force (kN), E (GPa) is the Young's modulus (210 GPa for steel), A is the cross-sectional area of the pile (m^2) and ε is the measured strain. By placing strain sensors at multiple levels, the distribution of force with depth can be ascertained, providing information on the local shear stresses mobilised within individual soil layers and hence enhancing knowledge of load transfer and stiffness under axial loading.

Traditional strain instrumentation for steel piles typically comprises vibrating wire (VW) or electrical resistance (ER) strain gauges which are attached to external shaft of the pile (Bica et al., 2014). Such instrumentation typically requires steel protection channels to be welded to the pile, as well as extensive cable management, to ensure survivability during impact driving. Whilst VW gauges are widely recognised for their long-term stability, their relatively slow response time limits their applicability to dynamic loading scenarios such as impact pile driving and signal drift may also result from inertial effects during the installation process (Bartz and Blatz, 2022). Improved response times are achievable with ER gauges, but their reliability is severely hampered by electrical interference and groundwater ingress (Flynn and McCabe, 2021).

The limitations of traditional instrumentation types can be overcome by using optical fibre Bragg grating (FBG) sensors (Kister et al. 2007). An FBG comprises a prefabricated grating within the core of an optical fibre which induces a periodic variation in refractive index to reflected light centred about the Bragg wavelength. When the sensor experiences a change in strain (due to mechanical and/or thermal effects), the Bragg wavelength undergoes a corresponding change, which can be correlated to strain. Their small size (<2mm in diameter), coupled

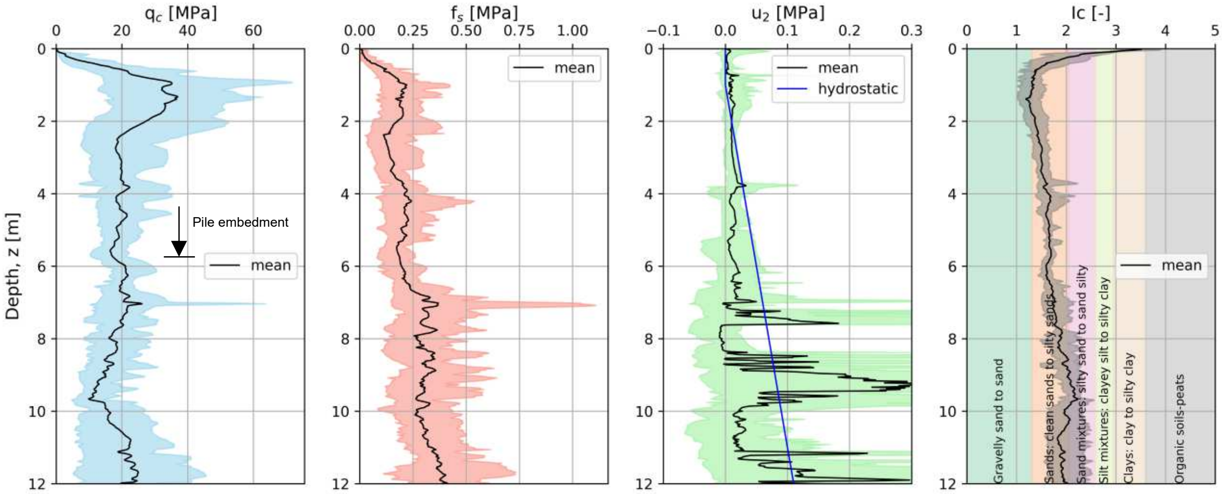


Figure 1. Cone penetration testing profiles showing mean (solid line) and min to max range (patch) within the axial pile test area

with the ability to house multiple sensors within a single continuous fibre string (known as multiplexing), promotes installation within small precut grooves on the external face of a steel pile without affecting the response of the pile (unlike traditional gauges). This approach was successfully utilised by Doherty et al. (2015) who demonstrated that FBG sensors can survive the driving process with remarkable stability. As such, FBGs have been routinely deployed in recent large-scale pile testing programmes such as PISA (Byrne et al., 2017) and, ALPACA (Buckley et al., 2023).

This paper discusses the installation and load testing of a 406 mm diameter open-ended steel tubular pile which was driven to a depth of 5.75 m in dense sand at Trinity College Dublin's geotechnical research site in Kilmuckridge, southeast Ireland. FBG sensors were used to provide insights into pile behaviour during installation and subsequent static tension testing to failure, with a particular focus on measurement of residual stresses induced in the pile during the installation process.

2 SITE DESCRIPTION

The test site is located within an active sand quarry near the village of Kilmuckridge, approximately 100 km south of Dublin, Ireland. The characterisation of the site is described in detail by Kasyap et al. (2026) and comprises a medium dense to very dense glaciofluvial sand with a mean effective particle size D_{50} of 0.18-0.41 mm and a mean uniformity coefficient C_u of 2.1-3.8. Shear box tests on reconstituted samples to a relative density D_r of 60% reported peak friction angles ϕ'_{peak} of 37° to 51° (depending on stress level) and a constant volume friction angle ϕ'_{cv} of ~33.5°.

The depth to groundwater at the test area was monitored continuously using probes in standpipe piezometers. The corresponding piezometric surface ranged from 0.3 to 1.6 m below ground level (bgl) over a 12-month period, with shallower levels corresponding to periods of heavy rainfall.

A total of 18 cone penetration tests (CPT) were performed within the pile test area to depths of 12 mbgl. Figure 1 presents typical profiles of cone resistance q_c, sleeve friction f_s, pore water pressure u_2 and soil classification index I_c with depth at the pile test location. These identify a layer of medium dense to dense sand with intermittent lenses of silt (identified by u_2 measurements below hydrostatic) to ~6 mbgl, with a very stiff sandy non-plastic silt present below this depth.

3 PILE TEST DETAILS

A series of instrumented test piles were installed at the site as part of a wider investigation into the effects of ageing on pile capacity in sands. The test pile discussed in this paper, referred to as *AM3*, comprised a 406 mm diameter, open-ended tubular steel pile (Grade S355) with a wall thickness of 9.5 mm and a total length of 6.3 m. To accommodate the FBG sensors, a 5 mm wide by 4 mm deep groove was milled into the external face of the pile on opposite sides, commencing 0.1 m from the base. Following cleaning of the grooves, the FBG sensor strings (containing 11 sensors each) were pre-strained to approximately 1200 με and glued into position using cyanoacrylate and then immersed in epoxy resin (Figure 2). Following curing, a layer of hot glue was applied over the top of the resin to provide a flush finish with the external surface of the pile for added protection during installation. Finally, the lead out cables were placed within PVC ducting, clamped to the pile and covered with a protective steel guard to prevent damage during transport and installation.

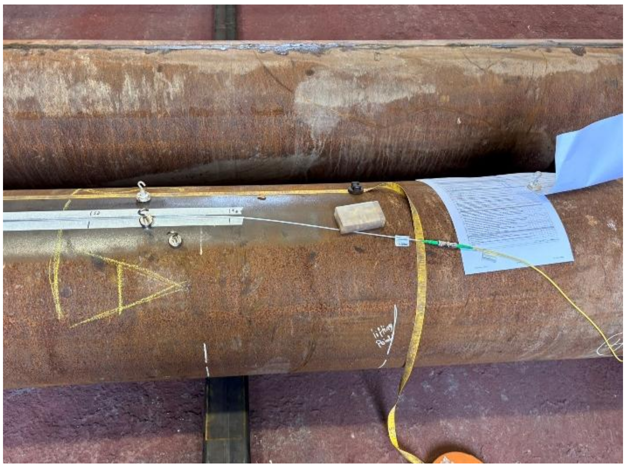


Figure 2. FBG sensor installation

The test pile was driven to a depth of 5.75 m using a Junttan SHK6 6-tonne hydraulic hammer. Due to the proximity of the lead out cables to the pile head, the piling contractor elected to use a steel dolly to prevent the hammer assembly from protruding over the cables, as shown in Figure 3. The drop height was limited to 200 mm to minimise compressive forces within the pile during driving. Driving was halted at regular intervals to measure the plug height using the procedure described by Gavin and Igoe (2021); the plug depth at the end of driving was 1.5 mbgl. Monitoring of the FBGs was carried out using a Luna si255 interrogator (Luna, 2025) at the maximum logging rate of 5 kHz to capture dynamic installation effects in a similar manner to Buckley et al. (2020). The sensors were zeroed when the pile was pitched into a vertical position but prior to seating of the hammer on the pile head. All sensors survived installation without issue.

Figure 3. Impact driving of Pile AM345

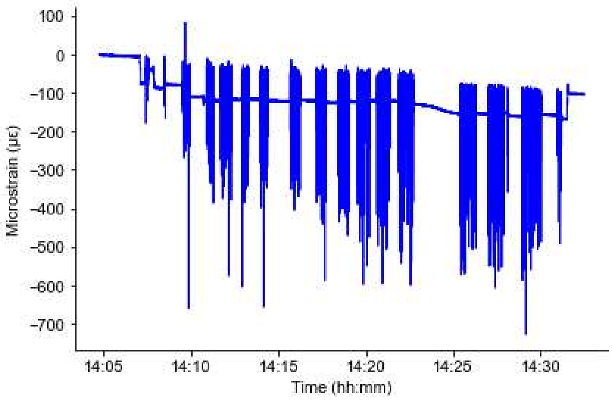
The pile was subjected to a monotonic tension load test approximately 20 days after installation. The load test setup, shown in Figure 4, comprised a 3 MN capacity reaction frame placed on two supporting beams at least 2 m from the test pile. The tension load was applied by pulling a 47 mm diameter threaded prestressed steel bar connected to the pile head by a welded steel circular plate. Pile head displacements were measured using 4 No. linear variable differential transformers (LVDT) connected to an independent reference beam. The static tension test utilised a fully automated load-controlled hydraulic system with a 200-tonne jack, which was programmed to maintain each target load level for a 10-minute period. A total of nine load increments were necessary to achieve pullout failure (based on excessive creep displacement under constant load), after which the pile was reloaded to failure once again to ascertain the ‘unaged’ tension capacity. The FBGs were re-zeroed immediately prior to commencing the tension load test and logged at a rate of 10 Hz.

Figure 4. Tension load test setup

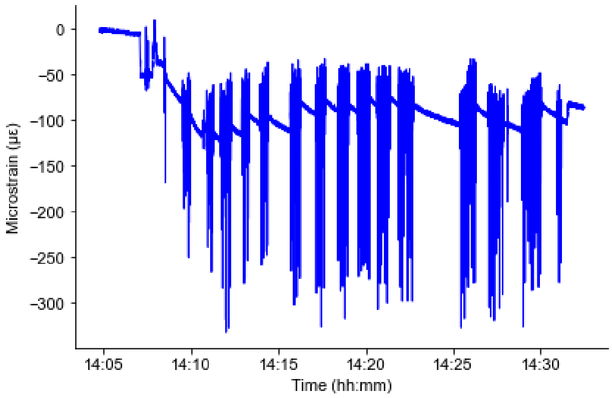
4 RESULTS

4.1 Installation

The variation in measured strain (uncorrected for thermal effects) with time during driving for Levels 2 and 10 (i.e. 1.0 and 5.0 m below ground level at end-of-driving) is shown in Figure 5(a) and (b), respectively. Note that negative strains in these figures pertain to compressive loads. The 5 kHz logging rate available using FBGs provides a unique insight into the behaviour of the test pile during impact, as well as pauses for plug measurement. Both levels exhibited an increase in compressive strains at the start of driving due to the weight of the hammer resting on the pile head. Continuous hammer impacts are characterised by peak compressive strains of up to 750 με at Level 2, reducing to between 200 and 450 με at Level 10 near the pile base which may be attributed to a combination of upward travelling stress waves reflected from the base (induced by previous hammer impacts), as well as shaft resistance generated between these levels.



(a)



(b)

Figure 5. Measured strain response with time during pile driving at (a) Level 2 and (b) Level 10

Pauses in driving to measure the plug height provided an opportunity to assess the accumulation of residual strains within the pile. At Level 2 (9.7 m above the pile base), residual strains were relatively constant during pause periods, as the sensors at this level remained above ground level for the majority of the drive and hence were not subjected to heat-induced friction. In contrast, compression strains were mobilised at Level 10 (0.7 m above the pile base) over the majority of the drive as the pile was restrained from rebounding by the soil. Furthermore, compressive strains tended to increase with time during stoppages. This behaviour, which was also observed by Doherty et al. (2015), is considered the result of dissipation of heat generated by friction with the surrounding soil during driving which induces thermal effects in the FBG sensors (although increases in radial stresses due to rapid dissipation of pore pressures within the sand may also be plausible). Strains were monitored for up to 1 hour after end of driving to allow the pile to reach equilibrium to provide a baseline for correction for thermal effects.

4.2 Tension load test

As noted previously, the FBG sensors were re-zeroed immediately prior to commencing the static tension load test. Figure 6 shows the corresponding strain response of the sensors at Levels 2 and 10 to monotonic tension loading. As expected, the tensile strains at Level 2 are greatest due to their proximity to the applied load at the pile head, with the corresponding response at Level 10 significantly reduced due to the generation of shaft resistance between these levels. Intermittent spikes in strain during each load hold is evident at all levels; this response is attributed to the automated loading system regulating hydraulic pressure to maintain the desired load. Minor deviations between opposing sensors at each level are also apparent due to non-uniform application of the applied load from the welded top plate. Finally, net tensile strains remain after unloading the pile, with the largest magnitude occurring at Level 10. As the pile was subjected to pullout failure, this observation implies that the pile was in a state of compression at the start of the tension test. This is explored further in the Section 5.

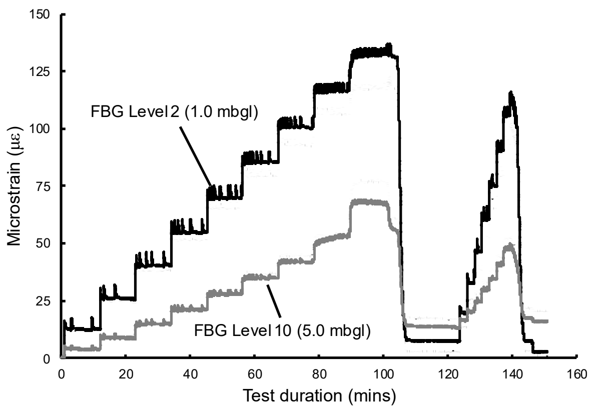


Figure 6. Variation in measured strain at Levels 2 and 10 with test duration during tension static loading

5 INTERPRETATION OF RESIDUAL LOADS

The term ‘residual load’ is typically used to describe the load developed in a pile between its installation and a subsequent load test. Whilst external sources of residual load such as negative skin friction may occur, the dominant cause of residual load in preformed displacement piles (i.e. steel or precast concrete) is the reversal of shaft resistance along the pile as it attempts to rebound after hammer impact during driving (Briaud and Tucker, 1984). Residual stresses can have a significant influence on the interpretation of pile load test data and hence need to be considered.

The ability of FBGs to provide stable, high-frequency readings make them desirable for measuring residual strains during driving. However, as highlighted in Section 4.1, these measurements are affected by thermal strains due to (i) differences in temperature of the pile and the ground, and (ii) heat generated by friction during driving. Heat dissipation may also be influenced by the presence of groundwater which may provide some form of cooling. Both the ambient air and ground temperatures (measured using the groundwater monitoring probes) at the time of driving were ~15ºC. Therefore, the generation of heat during driving would lead to expansion of the pile (and hence tensile strain) which would dissipate into the surrounding soil when driving ceases. As such, the reduction in strains during pause periods as shown in Figure 5(b) may be indicative of heat dissipation as the pile cools during the pauses in driving. Without explicit measurements of thermal strain, it is inherently difficult to assess the true residual stresses induced by pile installation. Furthermore, Buckley et al. (2020, 2023) note that full strain insensitivity with temperature-only FBG sensors is difficult to achieve in practice. As such, interpretation of strains during driving requires further work.

Residual stresses may also be evaluated using FBG strain data obtained from the static load test (which were re-zeroed at the start of the test). The final strain value recorded after unloading is taken as a reference offset which is then subtracted from the full strain dataset. Following this correction, the presence of negative initial strains in the strain–time response indicates the existence of compressive residual stresses within the pile. The resulting distribution of residual load with depth for all sensor levels (derived using Equation 1) is illustrated in Figure 7. The small tensile residual load measured near the pile toe is potentially anomalous as the residual force at the pile toe would be expected to be close to zero or slightly compressive. Nevertheless, the resulting residual load profile is in good agreement with previous observations for closed and open-ended piles reported in the literature (e.g. Igoe 2010; Paik et al. 2003). This profile may be used as a reference for assessing the residual loads derived from the FBGs at the end of driving, although additional residual loads arising from increases in radial stresses and ageing effects may also result in differences between the two interpretation methods.

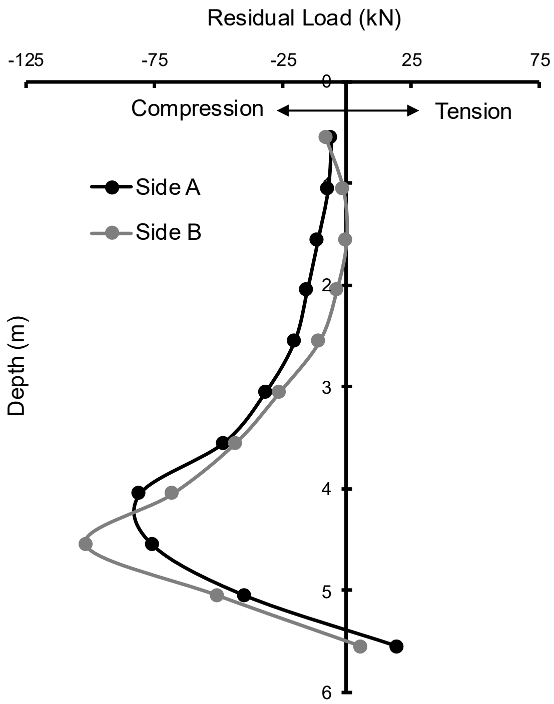


Figure 7. Distribution of residual load back-figured from measured strain response during tension load test

6 CONCLUSIONS

This study presents the strain response of an FBG-instrumented 406 mm diameter steel tubular pile during driving and maintained tension load test in fluvioglacial sand at the Kilmuckridge test site in Ireland. The following conclusions are drawn:

- Installation of the FBGs within pre-cut grooves provided adequate protection to ensure that the sensors survived impact driving without issues.
- Measured strains during impact driving were influenced by thermal effects (i.e. expansion and contraction) which was attributed to heat generated by interface friction. This was evident from changes in strain during intermittent pauses in driving to measure the plug height. Further analysis is required to determine suitable corrections for such effects.
- By re-zeroing the strain sensors immediately prior to commencing the load test, the residual strains at this point may be inferred by taking the final strain value recorded after unloading as a reference offset which is then subtracted from the full strain dataset. the resulting residual load profile is in good agreement with previous observations for closed- and open-ended piles reported in the literature.

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