Centrifuge and Real-Time Hybrid Testing of Tunneling beneath Piles and Piled Buildings

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Abstract: Tunnels are constructed increasingly close to existing buried structures, including pile foundations. This poses serious concerns, especially for tunnels built beneath piles. Current understanding of the global tunnel–soil–pile–building interaction effects is lacking, which leads to designs that may be overly conservative or the adoption of expensive measures to protect buildings. This paper presents outcomes from 24 geotechnical centrifuge tests that aim to investigate the salient mechanisms that govern piled building response to tunneling. Centrifuge test data include greenfield tunneling, pile loading, and tunneling beneath single piles and piled frames, all within sand. The global tunnel-piled frame interaction scenario is investigated using a newly developed real-time hybrid testing technique, wherein a numerical model is used to simulate a building frame, a physical (centrifuge) model is used to replicate the tunnel-soil-foundation system and structural loads, and coupling of data between the numerical and physical models is achieved using a real-time load-control interface. The technique enables, for the first time, the modeling of a realistic redistribution of pile loads (based on the superstructure characteristics) in the centrifuge. The unique dataset is used to quantify the effects of several factors that have not previously been well defined, including pile installation method, initial pile safety factor, and superstructure characteristics. In particular, the results illustrate that pile settlement and failure mechanisms are highly dependent on the pretunneling loads and the load redistribution that occurs between piles during tunnel volume loss, which are related to structure weight and stiffness. The paper also provides insight as to how pile capacity should be dealt with in a tunnel-pile interaction context. DOI: 10.1061/(ASCE)GT.1943-5606.0002003. This work is made available under the terms of the Creative Commons Attribution 4.0 International license, http://creativecommons.org/licenses/by/4.0/.

Introduction

The expansion of cities causes increased demand for underground infrastructure and the need to construct tunnels in the proximity of man-made assets. Excavation-induced ground movements and stress relief may adversely affect existing pile foundations and associated superstructures. In particular, tunneling beneath piles can result in differential pile settlements and, potentially, pile failure.

Research has provided empirical approaches, simplified analytical methods, and numerical analyses for the prediction of settlements and loss of capacity of existing piles due to tunnel excavation (Basile 2014; Devriendt and Williamson 2011; Jacobsz et al. 2004; Hong et al. 2015; Marshall 2012; Marshall and Haji 2015; Selemetas and Standing 2017; Soomro et al. 2015). However, few studies have recognized the importance of pile safety factor (Dias and Bezuijen 2015; Lee and Chiang 2007; Williamson et al. 2017b; Zhang et al. 2011) and the role of pile installation method (displacement versus nondisplacement piles). In addition, the impact of the superstructure action on tunneling-induced displacements of piles and the resulting deformations has received limited attention (Franza et al. 2017; Franza and Marshall 2018). These aspects require further investigation.

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Physical modeling using a geotechnical centrifuge has been used to investigate many of the mechanisms related to tunnel-pile foundation interaction (Jacobsz et al. 2004; Lee and Chiang 2007; Loganathan et al. 2000; Marshall and Mair 2011; Ng et al. 2013, 2014; Williamson et al. 2017a, b). Conventional centrifuge testing tends to break the problem down into discrete parts (e.g., isolated piles) and/or to use simplified superstructures (e.g., constant loads from the superstructure, rigid connections, and beams). However, a complete replication of soil-structure systems cannot be obtained when the structural and geotechnical domains are decoupled, and the use of a simplified superstructure limits the analysis and may significantly alter the soil-structure interactions.

Complex soil-structure interaction (SSI) problems can be studied through real-time hybrid models whereby the real-time substructure testing approach described by Blakeborough et al. (2001) is implemented: a physical test (modeling a key portion of the domain) and a numerical simulation (modeling the remaining domain) are run simultaneously, and shared boundary conditions are exchanged at a real-time frequency. Hybrid modeling of tunnel-building interaction using a reduced-scale centrifuge model is described in this paper and is referred to as coupled centrifuge-numerical modeling (CCNM). The feasibility of hybrid modeling using a geotechnical centrifuge has been demonstrated by recent pioneering works (Franza et al. 2016; Kong et al. 2015; Idinyang et al. 2018).

Scope of Work

This paper aims to illustrate the effects of installation method, initial safety factor, and load redistribution due to frame action on tunneling-induced settlements of axially loaded pile foundations and on the resulting superstructure deformations. Data are provided from a series of geotechnical centrifuge tests of tunnel...
excavation beneath piles and piled frames in dry silica sand using both conventional and CCNM test methods. All results are presented in model scale unless otherwise stated.

Test Details

A total of 24 tests were performed at 60 g (i.e., acceleration scaling factor $N = 60$) using the University of Nottingham Centre for Geomechanics centrifuge, as listed in Table 1, which provides details on pile load condition, geometry, and configuration. The tests are grouped into four categories: series A was a greenfield test; series B comprised pretunneling single pile loading tests (nondisplacement piles); series C investigated the response of isolated piles to tunneling; and series D modeled the response of piled frames to tunneling, applying the CCNM technique. The tests are labelled according to installation method (N = nondisplacement and D = displacement), pile position (Fig. 1), and initial safety factor (for example, N1SF1.5 represents a nondisplacement pile located in position 2 with an initial safety factor of 1.5); G indicates pile group and FR denotes the structural frame, which is discussed subsequently.

The layouts of series B, C, and D tests are presented in Figs. 1(a–c), respectively. In series C, displacement and nondisplacement piles in positions 1–3 were tested. Service loads were applied (kept constant with tunnel volume loss $V_t$) such that the initial safety factor $SF_0$ was either 1.5 or 2.5. Note that only piles

Table 1. Summary, in model scale dimensions, of centrifuge experiments

| Test series | Label (number of tests performed) | Pile type | Position number | Offset $x$ (mm) | Service load $P_0$ (N) | Capacity $Q_0$ (N) | $SF_0$ | Note |
|-------------|----------------------------------|-----------|-----------------|-----------------|-----------------------|-------------------|-------|------|
| A           | GF                               | —         | —               | —               | —                     | —                 | —     | —    |
| B           | LP                               | —         | —               | —               | —                     | —                 | —     | —    |
| C           | N1SF1.5 (1)                       | N         | 1               | 0               | 493                   | 740               | 1.5   | TPI  |
| C           | N1SF2.5 (1)                       | D         | 1               | 0               | 296                   | 740               | 2.5   | TPI  |
| C           | D1SF1.5 (1)                       | D         | 1               | 0               | 667                   | 1,000             | 1.5   | TPI  |
| C           | D1SF2.5 (1)                       | D         | 1               | 0               | 400                   | 1,000             | 2.5   | TPI  |
| C           | N2SF1.5 (1)                       | N         | 2               | 75              | 493                   | 740               | 1.5   | TPI  |
| C           | N2SF2.5 (1)                       | N         | 2               | 75              | 296                   | 740               | 2.5   | TPI  |
| C           | D2SF1.5 (1)                       | D         | 2               | 75              | 667                   | 1,000             | 1.5   | TPI  |
| C           | D2SF2.5 (1)                       | D         | 2               | 75              | 400                   | 1,000             | 2.5   | TPI  |
| C           | N3SF1.5 (1)                       | N         | 3               | 150             | 493                   | 740               | 1.5   | TPI  |
| C           | N3SF2.5 (1)                       | N         | 3               | 150             | 296                   | 740               | 2.5   | TPI  |
| C           | D3SF1.5 (1)                       | D         | 3               | 150             | 493                   | 740               | 1.5   | TPI  |
| C           | D3SF2.5 (1)                       | D         | 3               | 150             | 296                   | 740               | 2.5   | TPI  |
| D           | NGSF1.5FR00 (1)                    | N         | 1–4             | 0–225           | 500                   | 740               | 1.5   | TPGI |
| D           | NGSF1.5FR30 (1)                    | N         | 1–4             | 0–225           | 500                   | 740               | 1.5   | TPSI |
| D           | NGSF1.5FR50 (1)                    | N         | 1–4             | 0–225           | 500                   | 740               | 1.5   | TPSI |
| D           | NGSF1.5FR70 (1)                    | N         | 1–4             | 0–225           | 500                   | 740               | 1.5   | TPSI |
| D           | DGSF2.0FR00 (1)                    | D         | 1–4             | 0–225           | 500                   | 1,000             | 2.0   | TPGI |
| D           | DGSF2.0FR30 (1)                    | D         | 1–4             | 0–225           | 500                   | 1,000             | 2.0   | TPSI |
| D           | DGSF2.0FR50 (1)                    | D         | 1–4             | 0–225           | 500                   | 1,000             | 2.0   | TPSI |
| D           | DGSF2.0FR70 (1)                    | D         | 1–4             | 0–225           | 500                   | 1,000             | 2.0   | TPSI |

*A = nondisplacement piles; and D = displacement piles.

*The reported values do not account for the influence of the pile offset.

*TPI = tunnel-pile interaction; TPGI = tunnel-pile group interaction; and TPSI = tunnel-pile-structure interaction.

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Fig. 1. Test layout (in model scale): (a) loading tests; (b) tunneling beneath a single pile (showing locations of piles; only a single pile installed for each test); and (c) tunneling beneath a piled frame (using the CCNM technique).
in positions 1 and 2 had their tips within the influence zones, defined by Jacobsz et al. (2004), in which large pile settlements caused by tunneling are expected.

In series D, tests were performed for a transverse row (positions 1–4) of displacement or nondisplacement piles representing the foundation of a framed building. Four superstructures were considered, and only the structural stiffness was varied; superstructure weight was kept constant in all series D tests. The prototype superstructure consisted of an eight-story concrete frame \((E = 30 \text{ GPa})\) with story height \(h\) and span length \(S\) of 3 and 4.5 m, respectively. In Table 1, FR00 indicates a fully flexible frame, whereas FR30, FR50, and FR70 indicate frames of increasing stiffness with beam and column elements having square cross sections of \(0.3 \times 0.3\), \(0.5 \times 0.5\), and \(0.7 \times 0.7\) m², respectively. The frame was assumed linear elastic with fixed column-beam connections. The tests only considered vertical pile loads and settlements in the centrifuge, because these are the most influential parameters for this problem [as discussed by Franza et al. (2017)]. This implies a hinged connection between the pile cap and the base of a column, which does not necessarily reflect reality but has been found to have a secondary effect on results. For example, from the model of frame FR50 (detailed subsequently), the reaction force measured at position 1 \((R_{1,1})\) was obtained by varying the boundary conditions at positions 1–4 [Fig. 1(c)]. By imposing a settlement of 10 mm at position 1 while fixing all locations [vertical, horizontal, and rotational degrees of freedom (DOF)], a value of \(R_{1,1} = 634 \text{ kN}\) was obtained. By releasing the rotational DOF at positions 1–4, a value of \(R_{1,1} = 621 \text{ kN}\) was obtained (a 2.1% difference), and the changes in the vertical reaction forces at positions 2–4 were all less than 3%.

In test series D, initial service loads \(P_0\) could have been assessed for each pile using either a specific structural analysis accounting for the frame loading or the frame construction stages. However, the adopted approach (a uniform distribution of \(P_0\)) allows the isolation of the influence of the load redistribution due to building action without adding further complexity to the problem [e.g., the effects of varying pretunneling pile loads among the piles within a pile group on the tunnel-pile interaction (TPI) has not been investigated by previous research]. For the piled frame, building weight \(P_0\) resulted in \(SF_0 = 1.5\) and 2 for nondisplacement and displacement piles, respectively, which are in the range of typical design values.

**Coupled Centrifuge-Numerical Modeling**

This section presents the CCNM methodology (Franza et al. 2016; Idinyang et al. 2018) adopted in this study, as illustrated in Fig. 2(a). A numerical model simulates the superstructure, whereas the geotechnical domain (tunnel, ground, foundation, and structural loads) is replicated within the centrifuge. Shared boundary conditions were achieved using a real-time data acquisition and load-control interface (Idinyang et al. 2018), as illustrated in Fig. 2(b).

The CCNM methodology can be summarized as follows. First, the model piles are driven and/or loaded in flight with service loads \(P_0\). Second, the numerical model is started; physical and numerical models are coupled by the real-time interface through continuous and high-speed (a) transfer of incremental pile displacements \(u_t\) (measured in the centrifuge) to the numerical model, (b) retrieval of target loads \(P'\) obtained from the latest numerical simulation, and (c) adjustment of the pile loads \(P\) in the centrifuge to the target values \(P'\). Third, increments of tunnel volume loss \(\Delta V_{t,t}\), are induced in the model tunnel, causing pile settlements \(u_t\). Associated superstructure deformations result in pile load changes (i.e., \(P' \neq P_0\)). Increments of \(\Delta V_{t,t}\) are kept small and only applied once a stable state is achieved within the coupled centrifuge-numerical model. Franza et al. (2016) illustrated that this hybrid model is stable in flight and its load-control performance is satisfactory for this application.

**Centrifuge Model**

The centrifuge equipment is shown in Fig. 3 and is based on a tunneling model for plane-strain greenfield conditions (Zhou et al. 2014). A 90-mm diameter model tunnel buried at a depth of 225 mm (at axis) within dry sand was used to replicate a prototype 5.4-m diameter tunnel with 13.5 m of cover (cover-to-diameter C/D = 0.5). The sand was Leighton Buzzard fraction E, which has an average grain size of \(D_{50} = 0.12 \text{ mm}\), a specific gravity of \(G_s = 2.65\), maximum \((e_{\text{max}})\) minimum \((e_{\text{min}})\) void ratios of 1.01 and 0.61, respectively, and a critical state friction angle of \(\phi_c \approx 30°\). The tunnel comprised a cylindrical latex membrane filled with water. To replicate in-flight tunnel volume loss \(V_{t,f}\), a controlled volume of water was extracted from the model tunnel using a tunnel volume control system [comprising constant-head standpipe, solenoid valve, linear actuator, water-filled sealed cylinder, and linear variable differential transformers (LVDTs)] shown in Fig. 3(a). This process was conducted up to either pile failure or \(V_{t,y} = 10\%\). During the greenfield test, the GeoPIV image-based measurement technique was used to measure tunneling-induced soil displacements at the Perspex window (White et al. 2003).

For tests investigating tunneling beneath piles, a foundation consisting of either a single pile or four piles was used. All piles were located along the center of the container width. A view of the model pile and pile cap is illustrated in Fig. 4. A load cell was installed at each pile head to have a reliable measurement of the head load \(P\). LVDTs were used to measure pile settlements \(u_t\).

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**Fig. 2.** Coupled centrifuge-numerical model: (a) diagram of the coupling loop; and (b) decoupled geotechnical and structural domains.
Model piles were made from 12-mm diameter full section cylindrical aluminum rod with a total length of 185 mm and a 60° tip. A fully rough interface was obtained by bonding sand to the periphery of the piles, giving a final diameter \( d_p \) of 13 mm. A threaded hole was made in the top of the pile to allow attachment of a pile cap.

Pile caps were composed of two aluminum round connectors, a load cell, a plate for the LVDT armature, and a loading bar (Fig. 4). Each of the pile loading bars could be loaded/jacked independently using a four-axis servo actuator apparatus and lever system. Four L03 MecVel ballscrew actuators (MecVel, Bologna, Italy), shown in Fig. 3(a), with 100-mm stroke and 5-kN capacity (at 1 g) were used. As shown in Fig. 3(b), a heavy duty die-spring (stiffness rate \( 155 \text{ N/mm} \)) was placed inside each actuator cap to damp the pile load-actuator displacement relationship. The actuator caps were fixed to linear guide carriages to ensure vertical travel. Note that the loading system was only able to apply an axial load to the piles. The lever system illustrated in Fig. 3(c) was made using four aluminum beams and a frame to transfer the action of the actuators to the pile loading bars. Figs. 3(c and d) show the gantry used to hold the LVDTs and the guides used to ensure the verticality of the pile loading bars and prevent the transfer of bending moments by the lever system to the pile caps.

**Numerical Model**

A simple and computationally efficient matrix stiffness method structural analysis based on the finite-element method was performed in MATLAB to simulate the frame, adopting a first-order elastic analysis (i.e., the equilibrium analysis was performed for the undeformed configuration, and geometric nonlinearity, such as P-delta effects, were not considered). At prototype scale, the stiffness matrix \( K \) of the framed structure was obtained using Euler-Bernoulli beam theory for fixed column-beam connections. Hinged pile-superstructure connections were assumed to replicate the conditions in the centrifuge model (i.e., no bending moments transferred from superstructure to pile). Note that the CCNM methodology can accommodate more rigorous structural numerical analyses (e.g., considering structural damage/nonlinearity). The results presented here represent the first phase of testing of the...
developed CCNM application and provide important reference data for future testing that will explore the effects of structural nonlinearity.

**Real-Time Interface**

The real-time interface was designed to efficiently carry out the actuator control and data acquisition tasks [full details presented in Idinyang et al. (2018)]. It consists of a field programmable gate array (FPGA) controller for scalable hardware integration, interchangeable modules (for acquisition, relay triggering, motor control, and limit switch sensing), and a local computer that runs LabVIEW version 2015 SP-1. The FPGA controller and its hardware components were mounted on the centrifuge platform adjacent to the model container to minimize noise in the signals.

The main processes that couple the physical and numerical models are contained within two loops that are run independently and at different rates [Fig. 2(a)]. The LabVIEW program loop (on the PC) runs at a fixed time interval of 60 ms (17 Hz), which was found to be satisfactory for this application. This loop (a) monitors changes to the user interface, (b) gets centrifuge model sensor information from the FPGA, (c) feeds incremental pile settlements \( u \) to the numerical model (which runs as a component of the LabVIEW program using a MathScript node), (d) executes the structural analysis that computes new target loads, and (e) transfers the new target loads to the FPGA. The FPGA program loop, which runs at a real-time frequency (\( \approx 500 \) Hz), acquires data from the centrifuge sensors and adjusts the pile loads \( P \) in the centrifuge to the target values \( P^t \) by performing automatic load control using a proportional, integral, derivative (PID) algorithm.

To facilitate a range of centrifuge test requirements, the interface can switch between manual displacement-control mode (for pile installation) to execute extension/retraction of the actuators, and automatic load-control mode to actuate, through the PID control algorithm, either user-defined load demands or target forces \( P^t \) provided by the numerical model.

Finally, effective LVDT signal filtering was required to avoid unrealistic fluctuations of the target load \( P^t \). Signal noise from the centrifuge model is amplified by the scaling factor \( N \) in the data passed to the numerical model (simulating at prototype scale). Target load at model scale is \( P^t = K[N(u^c + u^o)]/N^2 \) for a given prototype structure with stiffness \( K \), where \( u^c \) and \( u^o \) are the model pile settlements and the error in the LVDT measurement due to signal noise, respectively (both at model scale). Target load fluctuation due to LVDT signal noise is \( P^{e} = Ku^e/N \). This aspect becomes more critical as superstructure stiffness \( K \) increases. The consequences of target load fluctuation is that the CCNM application can run into stability issues when analyzing very stiff superstructures; examples of this are presented subsequently.

**Model Preparation and Test Procedure**

Sand was placed by air pluviation to achieve a relative density \( I_d \) of 30%. Selection of this low relative density was based predominately on practical benefits related to sample preparation time; a short preparation time allowed for a large number of tests to be completed such that a comprehensive study of the mechanisms controlling the response of the piles (e.g., pile installation method, initial safety factor, and load redistribution within a pile group) could be accomplished. In addition, the greenfield tests [conducted in a way similar to the tests described by Marshall et al. (2012) and Zhou et al. (2014)] provided data that supplemented existing displacement data for dense \( (I_d = 90\%) \) and medium-dense \( (I_d = 60\%–75\%) \) sands and enabled a fuller understanding of the effects of relative density on tunneling-induced ground deformations (Franza et al. 2018). In series B–D, the test procedures can be summarized as follows:

1. After pouring the sand, the piles were installed at 1 g by jacking to the final embedment depth \( L_p \) for nondisplacement pile tests and \( L_p - 2d_p \) for displacement pile tests;
2. The model was spun to 60 g;
3. For pile loading tests (series B), piles were jacked while pile head reaction force and settlement were measured. For displacement pile tests, the piles were jacked in flight a distance of \( 2d_p \), and the pile head loads were subsequently reduced to the initial service value \( P_0 \). For nondisplacement pile tests, the service loads \( P_0 \) were directly applied to the piles. The value of the applied service load depended on the specified initial safety factor \( P_0 = Q_0/\text{SF}_0 \), where \( Q_0 \) is the pretunneling ultimate pile capacity and \( \text{SF}_0 \) is the initial safety factor). For pile groups, the piles were installed sequentially; the installation sequence of displacement piles started from pile 4 and moved toward pile 1, whereas the loading sequence of nondisplacement piles was from piles 1 to 4;
4. For tunneling beneath piled frame tests, the real-time interface was activated such that the applied loads \( P \) matched the numerical demand \( P^t \). For tests of single piles, the load demand was maintained constant during the entire tunneling process (i.e., \( P^t = P_0 \)); and
5. Small increments of tunnel volume loss (\( \Delta V_{10} \approx 0.02\%–0.04\% \)) were induced and pile settlements were measured. The adopted installation procedures, prior to tunneling, are able to capture the important features of nondisplacement piles (in which resistance is mainly provided by the shaft, because displacements are insufficient to mobilize base capacity) and displacement piles (in which, at the end of the installation, base capacity is mobilized and pile unloading results in negative shaft friction).

**Tunneling beneath Single Piles**

**Precut Load-Settlement Response**

The load capacity of the model piles \( Q_0 \) was required to evaluate the initial safety factor \( \text{SF}_0 \) of piles. For nondisplacement piles, \( Q_0 \) was assumed equal to the load required to push a pile a distance of 10% of the pile diameter \( d_p \). For displacement piles, \( Q_0 \) was evaluated based on the maximum force measured during the jacking of piles in position 2 (discussed subsequently).

The value of \( Q_0 \) for nondisplacement piles was assessed from the loading tests in series B, in which three tests were performed with the same configuration. The results, displayed in Fig. 5, show good repeatability, with an average \( Q_0 = 740 \) N at a settlement of 10%\(d_p\).

Fig. 6 displays the load-normalized settlement curves measured during the installation/loading of the piles in positions 1–3 during test series C. The results for the nondisplacement piles (upper subplots) show similar trends in the three different locations. The increase of the applied load resulted in greater pile settlement; however, settlement due to service loads were lower than 10%\(d_p\).

The displacement pile installations (lower subplots) highlight some interesting outcomes. Note that the reference value \( Q_0 = 1 \) kN (shown in Table 1) was measured from tests D2SF1.5 and D2SF2.5 (i.e., piles in position 2). Installations repeated at position 2 gave very similar results, illustrating good repeatability. However, the relative pile-tunnel location had an effect on \( Q_0 \) that was neglected by assuming a fixed reference value of \( Q_0 \). The value
of \( Q_0 \approx 1 \, \text{kN} \) is reasonable for piles in position 3 [tests D3SF1.5 and D3SF2.5, Fig. 6(c)]. However, piles in position 1 [tests D1SF1.5 and D1SF2.5, Fig. 6(a)] were clearly affected by the presence of the model tunnel. The piles in position 1 displayed a stabilization of the driving load followed by a decrease over the distance of \( 1 - 2d_p \). This unrealistic response occurred because the tips of the piles in position 1 were directly above and very close to the boundary of the water-filled model tunnel.

**Tunneling-Induced Settlement**

In drained conditions, tunneling can have a reducing effect on the capacity (\( \Delta Q < 0 \)) of nearby piles due to stress relief within the ground. An affected pile will move downward in an attempt to mobilize the forces (along the shaft and/or at the pile base) necessary to achieve equilibrium. Note that positive shaft friction is mobilized for small magnitudes of relative pile-soil displacements, whereas greater relative movements are needed to fully mobilize base resistance. Displacements will continue to occur as long as the mobilized capacity is lower than the pile load (\( Q < P \)). For a constant applied load \( P \), pile failure is initiated when capacity is reduced to the point where it matches the applied load (\( Q_{\text{max}} \rightarrow P \)), potentially inducing large pile movements (Jacobsz et al. 2004; Marshall and Mair 2011). Therefore, in a tunneling/geotechnical context, the term pile failure should describe the moment at which the rate of increase of pile settlement with \( V_{l1} \) shows a significant increase (this definition is adopted within this paper and is referred to as geotechnical pile failure). This framework implies that \( SF_0 = Q_0 / P \) influences pile failure. However, other failure criteria are also important to consider for practical and/or serviceability reasons (e.g., reasons related to the superstructure). The arbitrary full-scale displacement of 20 mm given by Jacobsz et al. (2004) may be used to define a threshold for large displacements (corresponding to 2.6%\( d_p \) for these tests), and the value of 10%\( d_p \) is used to refer to very large displacements (relating to the performance-based requirements of structures). These thresholds are indicated on subsequent figures for reference.

From test series C, normalized pile displacements (bottom subplots) are plotted against \( V_{l1} \) in Fig. 7, including shaded lines to indicate large and very large displacements. Greenfield displacements (black lines) at the pile heads and tips are also plotted. Pile displacements in position 3 generally fell within the range defined by the greenfield values at the pile tip and head. However, the piles in positions 1 and 2 diverged from the greenfield displacements from a very low \( V_{l1} \). The rate of displacement increased with \( V_{l1} \) for piles in positions 1 and 2, whereas it decreased for piles in position 3.

Pile initial safety factor \( SF_0 \) is shown to have a considerable influence on tunneling-induced settlement. In each position, the higher the value of \( SF_0 \), the lower the pile displacement for both nondisplacement and displacement piles. For \( SF_0 = 1.5 \), the displacement pile in position 1 failed suddenly at \( V_{l1} \approx 0.5\% \), whereas the nondisplacement pile did not show indication of failure. Displacement and nondisplacement piles in position 2 with \( SF_0 = 1.5 \) displayed a sharp increase in rate of displacement after \( V_{l1} = 1\% \) and 4\%, respectively. The data indicate that \( V_{l1} \) at geotechnical pile failure was higher for nondisplacement piles than for displacement piles for a given \( SF_0 \) and pile location. This was due to the fact that displacement piles rely on the highly stressed soil regions around their tips for capacity, which are more significantly affected by stress relief caused by tunnel volume loss (due to proximity) than the soil around the shaft [as illustrated by Franza and Marshall (2017) using cavity expansion/contraction analyses].

The data indicate that the very large settlement threshold (10%\( d_p \)) should not be used to define an ultimate geotechnical pile.
failure criterion. For instance, nondisplacement piles in positions 1 and 2 had an approximately constant rate of settlement with \( V_I_t \), even for vertical displacements greater than this threshold, indicating that a reserve of capacity was available. In these cases, the piles were simply moving with the tunneling-induced ground displacements, but at a higher rate than greenfield displacements due to the pile loads and the equilibrium condition that requires relative soil-pile settlement to remobilize capacity.

Overall, the data indicate that the risk of failure of isolated piles (with constant head loads) located within the tunnel influence zones defined by Jacobsz et al. (2004) (Fig. 1) is low for nondisplacement piles but may be an issue for displacement piles. Unfortunately, the acquired data does not enable an assessment of the post-tunneling safety factor of the piles. This useful information requires considerable testing effort, because a single test is required for each value of tunnel volume loss. This aspect will be the focus of future centrifuge testing at the University of Nottingham.

### Tunneling beneath Piled Buildings

#### Comparison between Greenfield and Pile Foundation Settlement

First, the response to tunneling of pile foundations subjected to superstructure weight but with a fully-flexible building (i.e., no load redistribution, FR00) was studied and compared with the greenfield data. Initial service loads \( P_0 \) were set equal to 500 N, giving \( SF_0 = 1.5 \) and 2 for nondisplacement piles but may be an issue for displacement piles. Unfortunately, the acquired data does not enable an assessment of the post-tunneling safety factor of the piles. This useful information requires considerable testing effort, because a single test is required for each value of tunnel volume loss. This aspect will be the focus of future centrifuge testing at the University of Nottingham.

The installation/loading of the row of piles showed good consistency between tests. The results for displacement piles were similar to those obtained during the installation of the single piles in series C. Nondisplacement piles in positions 2–4 generally behaved as the single piles did (refer to Fig. 6), reaching a final displacement of \( u_c/d_p \approx 5\% \). However, the nondisplacement pile in position 1, which was the first in the loading sequence, was affected by the loading of the adjacent piles. During its loading to 500 N, pile 1 reached a displacement of \( u_c/d_p \approx 5\%–7\% \) (similar to the response of pile 1 in the single pile load tests; Fig. 6). After loading the other piles, \( u_c/d_p \) of pile 1 reached 13\%, bringing it closer to the tunnel and making it more susceptible to the effects of tunnel volume loss. This additional displacement is referred to as interaction settlement and is discussed in more detail subsequently.

Further details on pretunneling installation/loading are given in Franz (2016).

Fig. 8 compares settlement from the greenfield test (GF) and tests for nondisplacement (NGSF1.5FR00, \( SF_0 = 1.5 \)) and displacement (DGSF2.0FR00, \( SF_0 = 2.0 \)) pile foundations with constant loads [i.e., a fully-flexible superstructure (FR00), which does not result in pile load redistribution during tunnel volume loss]. For clarity, only the displacement pile in position 1 is plotted for DGSF2.0FR00 (explained subsequently). The data show that nondisplacement piles 1 and 2 displayed settlement considerably larger than the greenfield scenario, whereas piles 3 and 4 settled only slightly more than greenfield surface values. The response of piles outside the tunnel influence zone (piles 3 and 4) did not agree with the outcomes of isolated piles in these positions, which settled less than the greenfield surface, as shown in Fig. 7 and indicated by Jacobsz et al. (2004). This must have been due to the group effect of the four piles. Finally, it is important to note that the nondisplacement pile group underwent failure, in which the settlement rate with \( V_I_t \) of pile 1 began to increases considerably at \( V_I_t \approx 1\% \), and an unstable condition occurred at \( V_I_t \approx 1.25\% \). Failure was not observed for pile 1 during the isolated pile test.
N1SF1.5 (Fig. 7). This disparity between the response of pile 1 in the isolated pile and pile row tests was a result of the difference in the pretunneling state of the ground in the two tests (i.e., because of multiple pile loading and interaction settlement in the pile row test).

The test with a row of displacement piles (DGSF2.0FR00) was terminated at $V_l; t \approx 0.2\%$ because of the brittle failure of pile 1. This agreed with the brittle failure of the isolated displacement pile test in position 1 with $SF = 1.5$, shown in Fig. 7.

To conclude, Fig. 8 illustrates that, for a given scenario, a critical response to tunneling of both displacement and nondisplacement pile foundations is predicted when constant loads are applied (i.e., a hypothetical fully-flexible superstructure). Furthermore, there may be a detrimental group effect on pile post-tunneling capacity; however, further research on this aspect needs to be undertaken given the uncertainties related to the effect of the model tunnel on the results.

Effect of Superstructure Stiffness

In this section, the effects of superstructure stiffness on tunneling-induced pile settlement and load redistribution are investigated. Fig. 9 shows the variation of the applied load in relation to $P_0$ (upper plots) and normalized settlements (lower plots) for piles 1–4 within the volume loss range $V_{l,t} = 0–3\%$. The change of force $\Delta P = P - P_0$ is referred to as the superstructure reaction force. Solid and dashed lines are used to indicate nondisplacement and displacement pile foundations, respectively, and a light-to-dark color transition indicates low to high superstructure stiffness. Note that tests for FR70 were interrupted at lower values of $V_{l,t}$ because they reached an unstable condition within the CCNM application as a consequence of the scaling of signal noise from the centrifuge to the numerical model.

There are two main outcomes that can be gleaned from Fig. 9: (1) the effectiveness of the superstructure in preventing pile failure; and (2) the complex pattern of load redistribution due to the superstructure. With respect to (1), although $\Delta P/P_0$ of pile 1 was lower than 10% and 20% for displacement and nondisplacement pile foundations, respectively, this decrease (due to the superstructure action) is able to prevent geotechnical pile failure, even for the relatively flexible frame FR30. Regarding (2), the superstructure stiffness redistributed the greatest portion of the total building weight (i.e., the initial load $P_0$) toward the pile in position 3, whereas the decrease in the load of pile 4 was greater than the reduction experienced by pile 1. Furthermore, it can be seen that the structural loads tended toward constant values at volume losses above about 2%–3% for the relatively stiff structures FR50 and FR70. These outcomes represent strong evidence of the need to account for the superstructure when undertaking a risk assessment related to tunnel construction beneath piled foundations.

Superstructure Deformation Mechanisms

To better understand the mechanisms responsible for load redistribution, pile head settlement profiles in the direction transverse to the tunnel axis are plotted in Fig. 10 at $V_{l,t} = 0.5\%$ and 1.0%. Greenfield vertical displacements at the locations of pile heads and tips are also displayed.

Comparison of the fully flexible (FR00) and rigid (FR70) tests allows identification of the contribution of superstructure stiffness
The magnitude of tunneling-induced settlements of piles depends considerably on the pretunneling pile safety factor $SF_0$, on the pile installation method (displacement/nondisplacement piles), and on the relative pile-tunnel location. Large pile settlements should be expected for both displacement and nondisplacement piles within previously defined tunnel influence zones (Jacobsz et al. 2004). The closer the value of $SF_0$ to unity, the greater pile settlements will be compared to greenfield values.

- Pile capacity in a tunnel-pile interaction context is generally not well defined. Failure of a pile affected by tunneling is most commonly based on displacement thresholds, which are certainly applicable; however displacement thresholds do not provide any indication of the actual change in pile load capacity or the post-tunneling pile safety factor. The data presented in this paper illustrated how initial safety factor and installation method influence the potential for geotechnical pile failure, which was defined as the tunnel volume loss where a sharp increase in pile displacement was observed. This failure occurs as the pile safety factor approaches unity (due to the effects of tunnel ground loss), and large pile displacements are required to mobilize the loads necessary to achieve equilibrium. The tests presented here did not evaluate the post-tunneling pile safety factor, an area of interest that will be explored in future research at the University of Nottingham. The topic of how pile capacity should be dealt with in a tunneling context (and indeed in other related problems such as deep excavations affecting nearby piles) requires further discussion and clarification within the engineering community.
- Installation method and $SF_0$ influence the potential for geotechnical failure of piles within the main tunnel influence area. For a given $SF_0$, volume loss at geotechnical failure of nondisplacement piles is greater than for displacement piles. If service loads are constant, failure is a critical aspect for displacement piles with a relatively low $SF_0$, whereas piles with $SF_0 \geq 2.5$ may not experience geotechnical failure even at high volume losses ($V_{lt} = 2\%–5\%$).
- Structural stiffness can effectively redistribute building loads among piles. A limited relative reduction in the pile load with volume loss can prevent geotechnical failure of piles directly above the tunnel. The frame action differs between displacement and nondisplacement pile foundations in terms of induced pile settlement. Building stiffness can significantly decrease the level of deformation of the buildings itself.
- A tunnel-single pile analysis with constant head load conditions (e.g., Marshall and Haji 2015) that neglects load redistribution due to structure action may result in an overly conservative assessment of global tunnel-piled structure interaction.
- The effectiveness of the real-time coupled centrifuge-numerical assessment of global tunnel-piled structure interaction.

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