
Peer reviewed version

License (if available): CC BY

Link to publication record in Explore Bristol Research

This is the accepted author manuscript (AAM). The final published version (version of record) is available via Society for Underwater Technology at https://sut.org/books-and-conference-proceedings/offshore-site-investigation-and-geotechnics-2023-conference-proceedings/. Please refer to any applicable terms of use of the publisher.

University of Bristol - Explore Bristol Research

General rights

This document is made available in accordance with publisher policies. Please cite only the published version using the reference above. Full terms of use are available: http://www.bristol.ac.uk/red/research-policy/pure/user-guides/ebr-terms/
A new load transfer model for axially loaded piles driven in chalk

K. Wen, S. Kontoe, R. Jardine

Imperial College London, UK

T. Liu

University of Bristol

ABSTRACT: Chalk poses geotechnical challenges for offshore foundation designers, because it can present as either a weak rock or a highly sensitive and putty paste, depending on the loading applied and the chalk’s variable natural structure. Piles supporting jacket structures mainly work through an axial push-pull action, while floating Tension Leg Platforms (TLPs) exert mainly axial tension. Their safe and economical design in chalk for dynamically sensitive offshore wind turbines is challenged by uncertainties regarding their axial load-displacement behaviour. This paper first introduces a recently developed load transfer model for axially loaded piles driven in chalk, which is expressed as a function of geotechnical properties measured directly by laboratory and in-situ tests, accounting indirectly for installation effects. The main aim of the paper is to assess the performance of the new model based on field pile tests at a well-characterized chalk site, in terms of axial displacements at the pile head, the distribution of axial force over the pile shaft and the profile of local shaft displacement at failure. Good agreement between the computed and measured pile head response demonstrates the applicability of the new proposed model and shows its advantages in comparison with existing industrial design approaches.

1 Introduction

Steel jacket structures founded on pin piles are commonly employed to support offshore wind turbines in difficult soils and water depths >35 m. The moments imposed by lateral wind, wave and current loads are carried by an axial push-pull action which generates compression and tension loading components on opposing pin piles. Floating TLPs, which may be deployed at deepwater sites, work primarily in axial tension. While the design of such foundations is driven primarily by the piles’ axial capacities under monotonic and cyclic loading, it is also vital to assess the piles’ axial load-displacement response to aid structural design and ensure the fixed or floating structures’ performance in service.

Chalk, a highly variable structured porous weak carbonate rock, is frequently encountered across large parts of northwestern Europe, extending from Poland to France and the British Isles (Jardine et al. 2019). It often poses particularly difficult geotechnical challenges for assessing axial pile performance. Reliable methods for assessing the axial capacity of piles driven in chalk are now emerging from recent research programmes including the UK Innovate and ALPACA Joint Industry Projects (JIPs) (see Jardine et al. 2019; 2023). However, no reliable method has yet been proposed to predict the piles’ axial load-displacement response.

The approaches taken to predict axial load-displacement for other geomaterials include numerical approaches (e.g. Chin et al., 1990; Lam et al., 2009), closed-form elastic analytical approaches (e.g. Randolph and Wroth, 1978; Crispin et al., 2018; Niazi and Wangensten-Øye, 2020) and Winkler-based load transfer approaches (e.g. Coyle and Reese 1966; McCabe and Lehane, 2008; Abchir et al., 2015).

Coyle and Reese (1966) first proposed a load transfer approach for pile shaft, also known as the ‘t-z’ curve model which characterises the mobilisation of local shaft resistance (τs) as a function of local shaft displacement (wz). Since then, researchers have proposed empirical and analytical extensions to cover various soil types, and bored or driven pile installation procedures. Accurate ‘t-z’ models provide tools for assessing axial pile performance, however, most of the ‘t-z’ curves proposed in the literature cover piles in sands and clays and their applicability to piles driven in chalk is uncertain and untested.

This paper introduces a ‘t-z’ model for piles driven in chalk which relies on fundamental chalk properties derived from laboratory and in-situ tests. Since pile installation causes extensive damage to the natural chalk mass, it is necessary to divide the chalk mass into specific chalk zones with distinct properties.
(Jardine et al. 2019, 2023). The analysis of their combined impact takes advantage of the analytical framework initially developed by Randolph and Wroth (1978). The proposed ‘r-z’ model is verified against field tests on a large diameter open-ended driven steel tubular pile tests reported by Jardine et al. (2023) at the well-characterised St Nicholas-at-Wade (SNW) chalk site employed by the ALPACA Plus JIP. The new method captures the piles’ strongly nonlinear behaviour well and allows more reliable load-displacement analyses for offshore piles driven at chalk sites.

2 Methodology

2.1 Pile installation effects

Field observations have confirmed that pile driving de-structures and damages the chalk around driven piles and leaves two distinct Zones (A and B) that suffer different degrees of damage. Figure 1 shows a schematic description of the chalk zones identified around pile driving at SNW, based on observations from Muir Wood et al. (2015) and Buckley et al. (2018).

- Zone A: an annular zone of remoulded chalk. No obvious gaps exist between this zone and the pile shaft. The thickness of this zone, \( t_{\text{putty}} \), is influenced by the pile section type and installation method. Muir Wood et al. (2015) reported that the thickness of the puttyed zone is around 0.78 to 1.57 times the pile wall thickness \( t_{w} \) and keeps constant with soil depth, while Buckley et al. (2018) found out that the puttyed zone thickness could approximately vary from 0.59 \( t_{w} \) to 1.64 \( t_{w} \).
- Zone B: intact chalk where markedly greater-than-natural fracturing is caused by pile driving. Slight heave can also be observed after pile driving. The Zone within which fracture spacings are reduced very significantly extends to radial distances of 6\( t_{w} \sim 11\ t_{w} \) beyond the pile wall;

The chalk mass, in Zone C, outside these damaged regions, remains undisturbed. Although the chalk at the SNW test site is naturally fractured, and has fracture apertures up to 3 mm wide, these fractures are far more widely spaced in Zone C than in Zone B.

The chalk in Zones A and B has markedly lower shear strengths and stiffness to that in Zone C, making it necessary to take these regions into account separately, basing their properties on high-quality laboratory and field testing.

2.2 Shearing behaviour of puttyed and intact chalk

Extensive laboratory testing on chalk samples was undertaken for the ALPACA project (Vinck et al. 2022). The low- to medium-density chalk encountered at SNW exhibits a high liquidity index and can be reduced to a very soft putty (with undrained shear strength \( S_{u} < 10 \text{ kPa} \)) by dynamic compaction at its natural water content. The laboratory research included suites of undrained triaxial tests on locally instrumented de-structured chalk described by Liu et al. (2022) in which putty samples were consolidated to isotropic mean effective stresses \( p_{0} = 70, 200 \) and 400 kPa and were then allowed to age under drained conditions until creep rates slowed to negligible values. Figure 2 presents the measured shearing stress-strain response from these triaxial tests, as determined from high resolution local strain sensors. The samples’ effective stress paths confirm that they behaved first in a ductile manner before undergoing phase transformation points at the shear strains \( \gamma = e_{a} - e_{r} \) around 1.0% to 2.0% under these three pressure levels. Loading to larger strains led to a strongly hardening behaviour and restrained dilation.

![Figure 1. Schematic diagram of the various chalk zones created by pile installation](image)

![Figure 2. Shearing stress-strain behaviour of the puttyed chalk from triaxial undrained tests (Liu et al., 2022)](image)
Figure 3 shows the degradation of equivalent isotropic secant stiffness $G_{\text{putty}} = E_d/3$ derived from vertical undrained Youngs moduli with shear strain. The elastic stiffness observed at very small strains defines the maximum initial stiffness $G_{o,\text{putty}}$. Wen et al. (2023) suggest a power-law function to capture the relationship between $G_{o,\text{putty}}$ and $p_0'$, based on the results of resonant column tests conducted by Fugro (2012) and locally instrumented triaxial tests carried out at Imperial College. This relationship captures the shear stress-strain behaviour of puttified chalk that has been normally consolidated after its formation around driven pile shafts to higher stresses by the in-situ set-up processes, as described by Jardine et al. (2023).

Wen et al. (2023) propose a simplified, empirical constitutive expression to describe the shear stress-strain response of the putty chalk at OCR = 1, based on the triaxial test results. The contractive and dilative phases of behaviour are covered by different mathematical formulations. The main input parameters include $G_{o,\text{putty}}$, the shear stress $\tau_{PT}$ at Phase Transformation Points (which increases with $p_0'$) and four nondimensional fitting parameters calibrated from undrained triaxial tests. Further details regarding the formulation are given below, while Wen et al. (2023) describe the model development.

If $\tau < \tau_{PT}$,

$$Y_{\text{putty}} = \frac{\tau}{G_{o,\text{putty}} \left[1 - \lambda_d \left(\frac{\tau}{\tau_{PT}}\right)^{\lambda_b}\right]}$$  \hspace{1cm} (7a)

If $\tau \geq \tau_{PT}$,

$$Y_{\text{putty}} = \frac{\left(\tau - \tau_{PT}\right)^{\lambda_d}}{\lambda_c} + \frac{\tau_{PT}}{(1 - \lambda_d) G_{o,\text{putty}}}$$  \hspace{1cm} (7b)

Where $\lambda_a, \lambda_b, \lambda_c, \lambda_d$ are model parameters calibrated from triaxial tests.

Figure 3. Variation of secant $G_{\text{putty}}$ with shear strain.

Figure 4 compares the predicted and measured shear stress-strain curve for samples taken to $p_0' = 200$ kPa, covering the $\leq 3\%$ and $\leq 25\%$ strain ranges. The model captures the shearing response well over the pre- and post Phase Transformation ranges.

Considering the Zone C chalk, Vinck et al. (2022) reported that the in-situ shear modulus profile $G_{o,\text{intact}}$ is influenced significantly by fissuring and to some extent by flint layering. Vinck (2021) also showed that the geophysical shear wave velocities could be correlated with the chalk’s CPT corrected tip resistances $q_t$.

Jardine et al. (2023) and Pedone et al. (2023) also show that the stiffnesses inferred from both laboratory and field shear wave velocities greatly overestimate the operational stiffnesses of the undisturbed natural chalk mass under even very low loads. They ascribe this mismatch to the natural chalk’s systems of micro-to-macro fissures. Their analyses showed that the mass operational elastic shear modulus of the intact chalk $G_{o,\text{intact}}$ is only $\frac{1}{4}$ of the geophysical small-strain shear modulus $G_{\text{max}}$. Their finding has been incorporated into the present study to capture the effect of fissuring within the natural chalk mass.

3 Development of the new shaft $t$-$z$ model

Approximating the chalk mass resistance developed around an axially loaded pile as coming from a series of concentrically shearing cylinders, the distribution of shear stress $\tau$ within the chalk mass can be estimated by a decay trend with the radial distance from the local maximum value applied at the pile shaft (Randolph and Wroth, 1978), as expressed by:

$$\tau = \tau_s \left(\frac{r_s}{r}\right)$$  \hspace{1cm} (1)
Where \( \tau_s \) = shear stress on the pile-soil interface (local shaft resistances), \( r \) = radial distance from the pile shaft center, \( r_o \) = pile outer radius.

Neglecting any radial soil displacement caused by Poisson straining of the pile material, the vertical displacements of the soil can be obtained at any radial location by integrating the shear strains over the distance from that location to a hypothetical far-field boundary \( r_m \) beyond which the effect of axial loading is considered negligible (see Figure 1). The estimate of \( r_m \) is discussed in the following section.

Prior to the slip taking place between the local pile shaft and the adjacent chalk, the local pile shaft displacement can thus be represented as:

\[
w_s = \int_{r_o}^{r_m} \gamma dr = \int_{r_o}^{r_o+\varepsilon_{putty}} \gamma(\tau) \, dr + \int_{r_o+\varepsilon_{putty}}^{r_o+\varepsilon_{putty}+\varepsilon_B} \gamma(\tau) \, dr + \int_{r_o+\varepsilon_{putty}+\varepsilon_B}^{r_m} \gamma(\tau) \, dr + \int_{r_o+\varepsilon_{putty}+\varepsilon_B}^{r_o+\varepsilon_{putty}+\varepsilon_B+\varepsilon_B} \gamma(\tau) \, dr \tag{2}
\]

Where \( \gamma \) = shear strain in chalk mass; \( \varepsilon_{putty}, \varepsilon_B \) = thickness of Zone A and Zone B respectively.

In order to obtain an explicit formulation of Equation 2, the following assumptions are made:

- The putttied chalk zone adjacent to the pile shaft is thin, so the average shear stress and strain levels at the midpoint of Zone A can be taken as applying over the full annulus, which facilitates the integration.
- The explicit modelling of Zone B is uncertain due to the unclear impact of fissure spacings on the operational shear modulus of the chalk. Pedone et al. (2023) stress the importance of Zone B when analysing piles under lateral loading and found it necessary to reduce the shear moduli within this Zone by an order of the magnitude from the geophysical values. However, multiple analyses undertaken by the Authors showed that the Zone B properties have far less impact on axial loading than those in Zone A and showed that it is acceptable to assign similar stiffnesses in Zones B and Zone C, as the axial stiffness is controlled primarily by Zones A and C.
- The intact chalk mass in Zone C remains within its operationally linear elastic range throughout the loading.

Based on the above assumptions, the developed shaft ‘I-z’ model is expressed mathematically as set out below:

If \( SR < \tau_{s,c}/\tau_{s,f} \),

\[
\frac{w_s}{r_o} = \frac{\varepsilon_{putty}}{r_o} \cdot \left( \frac{\eta \tau_{s,f}}{G_o, putty} \right) \left[ 1 - \lambda_a \left( \frac{\eta \tau_{s,f}}{\tau_{s,c}} \right) \right]^{-1} + \zeta_C \tag{3}
\]

Where \( \tau_{s,c}/\tau_{s,f} \leq SR < 1 \),

\[
\zeta_C = \frac{1}{r_o} \int_{r_o}^{r_m} \gamma(\tau) \, dr = SR \cdot \left( \frac{\tau_{s,f}}{G_o, intact} \right) \cdot ln \left( \frac{r_m}{\tau_o+\varepsilon_{putty}} \right) \tag{4}
\]

Where \( SR = \) the local shaft resistance ratio \( = \tau_{s,c}/\tau_{s,f}; \eta = r_o/(r_o + 0.5\tau_{s,f}) ; \tau_{s,f} = \) ultimate local shaft resistance that are assessed independently. It is assumed that the strength of Zone A can be modelled on the basis of isotropically consolidated laboratory tests. To avoid the need to separately assess the effects of pile installation and set up on the shaft stresses, the initial local effective radial stress \( \sigma_\tau' \) prior to loading is adopted as the radial effective stress \( \sigma_\tau' \) at failure predicted by the ALPACA-SNW pile axial capacity design approach, neglecting any normal stress changes provoked by shaft loading (Jardine et al., 2023). The \( \lambda_c \) and \( \tau_{s,c} \) terms can then be expressed as follows:

\[
\lambda_c = 732 \cdot (\sigma_\tau'/p_0) \tag{5}
\]

\[
\tau_{s,c} = \tau_{pr} \eta^{-1} = 0.26 \sigma_\tau' \eta^{-1} \tag{6}
\]

Where \( p_0 = \) Reference atmosphere pressure.

As noted earlier, a hypothetical upper-bound radius, \( r_m \), is defined beyond which the influence of pile axial loading is deemed negligible. Randolph and Wroth (1978) suggested that \( r_m \approx 2.5L(1 - v) \) where \( L \) is the pile embedment length, and \( v \) is the Poisson’s ratio. Adopting these boundary conditions, the intact chalk term \( \zeta_C \) in Equation 3 and Equation 4 can then be expressed as:

\[
\zeta_C = \frac{1}{r_o} \int_{r_o}^{r_m} \gamma(\tau) \, dr = SR \cdot \left( \frac{\tau_{s,f}}{G_o, intact} \right) \cdot ln \left( \frac{r_m}{\tau_o+\varepsilon_{putty}} \right) \tag{7}
\]

Figure 5 illustrates conceptually the two branches of the model, in conjunction with the local ultimate shaft resistance \( \tau_{s,f} \) given by the Jardine et al. (2023) method. The new model represents the effect of pile installation implicitly and is developed based on the in-situ and laboratory chalk properties. The local shaft displacement corresponding to failure \( w_{s,f} \) is associated to the pile diameters and \( \tau_{s,f} \) values (Wen et al. 2023). Once \( w_{s,f} \) is exceeded, slip takes place between the pile local shaft and the adjacent chalk and \( \tau_{s,f} \) is assumed to remain constant. More details regarding the development of the ‘I-z’ model are given by Wen et al. (2023).
Figure 5 also shows the conventional ‘t-z’ models of API (2014). A typical mean relationship $w_{s,f} = 0.01D$ (where D is the outside pile diameter) is recommended for routine design purposes, although field tests suggest an extreme possible range between 0.0025D and 0.02D that may be explored in cases where axial pile stiffness is crucial to safe design. The ALPACA axial field tests on piles driven in chalk do not show any need to incorporate a post-peak ‘falling branch’ section as incorporated into the API ‘t-z’ model for clay as shown in Figure 5.

$$Q_{tn} = \tau_{s,n} \cdot L_n \cdot \pi D + Q_{b,n} \quad (10)$$

(6) Substitution of Equation 10 into Equation 8 and re-computation of the elastic deformation $w_{e,n}$ follows. The midpoint movement of the pile segment $w_{m,n}$ is also updated.

(7) Steps (2)-(6) are repeated until the $w_{m,n}$ values of segment $n$ converge within a set tolerance. The force and displacement at the top of segment $n$ is then assumed to act as the boundary condition at the bottom of the upper segment, $n-1$.

$$Q_{b,n-1} = Q_{tn} \quad (11)$$
$$w_{b,n-1} = w_{t,n} = w_{b,n} + w_{te,n} \quad (12)$$

Steps (3) - (7) are repeated until $w_{t,1}$ and $Q_{t,1}$ is solved iteratively at the pile head segment and stored. The segments in the region of the pile stick-up should also be considered, taking zero side resistances.

The above procedure is then repeated using different assumed pile tip movements $w_{b,n}$ until a series of force and displacement pile head points are obtained and plotted as a computed pile load-displacement curve. Analyses may be completed within a few minutes on a standard PC, depending on the number of soil layers considered.

Figure 5. Conceptual sketch for the developed ‘t-z’ model and API model (2014)

4 Algorithm for calculating axial load-displacement response at pile head

Figure 6 provides the flow chart covering the Authors’ implementation of the new load transfer model in MATLAB, which allows the prediction of the load displacement curves at the pile head.

(1) The pile’s total length $L$ is subdivided into $n$ segments depending on the site-specific soil stratigraphy. The pile segments may have different lengths $L_n$ and are labelled from the pile top down to its tip.

(2) Pile tip movements $w_{b,n}$ developed at the bottom base segment $n$, provoke base resistances $Q_{b,n}$ under compression which are computed from a parallel ‘Q-z’ model. In pile tension cases, $Q_{b,n} = 0$.

(3) Assuming an axial force $Q_{tn}$ at the top of segment $n$ and a linear variation of load in the segment, the corresponding elastic pile deformation $w_{te,n}$ at the same position is calculated as:

$$w_{te,n} = \frac{(Q_{b,n}+Q_{tn})L_n}{2E_{steel}A} \quad (8)$$

(4) The displacement at the midpoint of segment $n$, $w_{m,n}$, is obtained by summing the initial assumed $w_{b,n}$ and one-half of the $w_{te,n}$.

$$w_{m,n} = w_{b,n} + \frac{w_{te,n}}{2} \quad (9)$$

Figure 6. Flow chart for applying ‘t-z’ models to predict the axial loading-displacement response at pile head
5 Prediction of pile performance to axial loading

To assess the performance of the developed load transfer model, a comparison is presented with the field data from the test on steel tubular pile, TP1, which was the largest driven pile for the ALPACA and ALPACA Plus JIPs (Jardine et al. 2023).

The SNW research site is a disused quarry located near Margate in NE Kent, UK, where the low-to-medium density chalk has CIRIA grade B2/B3, and its engineering properties are summarized by Buckley et al. (2018) and Vinck et al. (2022). The site was investigated by extensive campaigns of cone penetration tests (CPT) as well as seismic downhole, crosshole and SCPT tests. Figure 7 shows the profile of CPT measurements and the shear moduli inferred from vertical shear wave velocities, indicating that \(q_t\) generally falls between 10 MPa to 20 MPa and the shear modulus slightly increases with chalk depth.

TP1 pile, which was comprehensively instrumented with FBG strain gauges, had an outside diameter of 1.8 m, a wall thickness of 0.025 m and was driven to 18 m embedment. It was left with a stick-up length above ground level to facilitate its single axial test, which took place after 373 days of undisturbed ageing after driving. A load cell and displacement transducers attached to the pile head recorded its response to slow, maintained load, tension testing. Figure 8 compares the measured load-displacement response with the prediction made using the new load transfer model through the abovementioned MATLAB code. The pile shaft was discretised into a series of horizons to capture the variations of shaft resistances calculated by applying ALPACA-SNW approach with sufficiently fine sub-divisions to match the overall pile axial capacity accurately. Good agreement can be seen between the calculated and computed load displacement responses, supporting the applicability of the proposed ‘\(r-z\)’ model. However, the match achieved is affected by creep displacements near failure, which are not captured by the proposed model. Also shown in Figure 8 are predictions from two commonly used ‘\(r-z\)’ models for sands given by API (2014) and DNV (2016). While the API curve significantly overestimates the axial displacements, the DNV ‘\(r-z\)’ model produces more comparable predictions to the proposed ‘\(r-z\)’ model over the initial loading stages, although it under-estimates movements at high loading levels.

Figure 9 compares the calculated and measured axial force distribution at various pile head loads (\(Q_t\) = 2504 kN, 4545 kN, 5740 kN). The proposed model generates a reasonable match to the experimental data, with only marginal differences appearing near the pile tip. The axial forces inferred from the FBG strain gauges show some inevitable experimental scatter, as reported by Jardine et al. (2023). The gradients of the smoothed curves reflect the mobilized axial shaft resistances. Most resistance is developed over the upper half of the pile initially. However, as \(Q_t\) increases, failure progresses downwards, and engages the shaft resistance of the lower pile sections.
The pile head displacement can be broadly split into two components: pile rigid motion and pile elastic compression or, in this case elastic stretching. Figure 11 shows the relative contribution of the pile’s elastic stretch in two different cases, as given by the newly developed ‘t-z’ model. Pile stretch plays a key role throughout loading; under lower axial loads, the pile head displacement was mainly due to the stretch of the pile’s top half, and this is why the predictions given in Figure 8 appear insensitive to the ‘t-z’ model assumed. As axial loads \( Q_t \) increase, the pile’s role in assisting progressive failure becomes more important and defines how shaft resistances are mobilised down to the pile tip.

Further work is in hand to model the impact of loading rate and constant load pause periods on the axial response of piles driven in chalk.

6 Conclusions

This paper introduces a newly developed ‘t-z’ model for driven piles in chalk, which considers pile installation effects through the division of the chalk mass into three distinct domains. The main conclusions drawn are:

1. Non-linearity in the stress-strain response of the puttified chalk may be represented faithfully through parameters calibrated from laboratory and in-situ tests. Stiffness assessment within the mass of undamaged chalk had to account for the chalk’s natural fissuring.
2. The proposed model gave good predictions for an exemplar open-ended pile tested in tension at the ALPACA research site.
3. The successful predictions included overall load-displacement response as well as axial force distribution and the profiles of local shaft displacement at failure.
4. While local pile shaft displacements are initially controlled by the pile’s compressibility and intact chalk’s elastic stiffness, the annulus of puttified chalk formed around the shaft during driving plays a progressively more important role as the loads grow towards failure.

The proposed model is more extensively explained and tested by Wen et al. (2023) which consider eleven other field tests on piles with diameters from 0.14 m to 1.37 m across multiple chalk-dominated sites. Overall, the newly proposed model offers better accuracy than the application of existing ‘standard’ sand and clay ‘t-z’ models.

7 Acknowledgements

The authors wish to acknowledge the critical contributions of Drs Ken Vinck and Roisin Buckley and other researchers and technicians at Imperial College London and Socotec Ltd who contributed to the field pile and laboratory testing and shared their data. The first author acknowledges the financial support of a Skempton scholarship of the Department of Civil &
Environmental Engineering at Imperial College and support from the China Scholarship Council (CSC).

8 References


