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Design Parameters for Shaft Resistance of Driven Monopiles and Suction Buckets from Rate-Dependent Interface Tests

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ABSTRACT: This paper describes how interface tests combined with pore water pressure measurements (IS-P), can be used to calculate the shaft capacity of offshore monopiles and suction buckets. A novel interface shear apparatus equipped with pore water pressure measurement was developed for conducting interface tests on samples from three different soil types. The β values used for calculating skin friction for piles were derived from the test results. Effective stress parameters were solely used for the calculated β values. Moreover, the results demonstrate rate dependence of interface strength and the corresponding β values. The rate dependence is also evident in the magnitude of the excess pore water pressure produced. Additionally, the pore water pressure values displayed different types of time dependency depending on the rate of loading. Consequently, the calculated effective normal stress values were influenced accordingly. These phenomena exhibited varying effects depending on the type of soil. Studies suggest that variations in rate effect along the pile length may be influenced by the soil's stress state. In general, β values ranging from 0.018 to 0.51 were calculated from the conducted IS-P tests. It was observed that IS-P tests can serve as a valuable supplementary method for determining parameters essential for analysing pile shaft capacity.

Keywords: Rate dependence, IS-P, interface strength, shaft resistance, and the β - method

1 INTRODUCTION

A considerable part of the axial capacity of offshore monopiles and suction buckets is determined by the skin friction of the pile shaft. Thus, an enhanced design method for estimation of shaft capacity is necessary to ensure both the short-term and long-term stability of monopiles and buckets. The conventional pile design method (α) is based on undrained shear strength, which is empirically linked to both in-situ and laboratory tests. These tests include cone penetration tests, in situ vane shear test, unconfined compression test and triaxial test (Esrig and Kirby, 1979; Burland and Twine, 1988). However, the undrained shear strength of remolded fine-grained soils can vary considerably depending on the sampling and testing methods used (Karlsrud et al., 2005). Consequently, there exists a considerable gap between the predicted and measured pile capacities. Some studies report that the predicted shaft resistance values ranged from 50% to 150% of the measured capacities obtained from pile load tests (Poulos H.G., 1989). Moreover, recent studies have shown that further gains in the axial capacity of piles are observed with aging (Jardine and Chow, 2006). Additionally, the development of offshore foundations into more advanced pile-type anchors requires improved laboratory methods to better determine shaft resistance. The magnitude and direction of skin friction for tension pile anchors differ from those for monopiles and suction buckets.

The β method was developed to estimate the shaft capacity of piles based on effective stress parameters. The effective vertical normal stress divided by the measured shaft resistance from pile load tests is used to calculate the β values (Jegandan et al., 2012; Karlsrud et al., 2005; Jardine et al., 2005). However, this method is an empirical approach and its applications are limited to site specific conditions. The application of the β method in most cases is limited to granular soil types. Based on previous researches, there are different empirical and experimental methods to calculate the β values (Burland, 1973; Karlsrud et al., 2005; Martinez and Stutz, 2024). Notably, Burland, (1973) derived correlations for the β values based on the results he obtained from Pile load tests. Figure 1 shows a comparison of different equation for calculating the β-values (Burland, 1973).

Efforts were made to establish correlations between laboratory interface tests and pile shaft resistance. In this context, Martinez and Stutz (2024) conducted interface shear tests with pore water pressure measurements to determine the β values. Other studies have focused on calculating the interface friction angle between various interface materials and soil types (Burland, 1973; Jardine et al., 1993). In particular, Jardine et al. (1993)

demonstrated that operational values of the interface friction angle (δ_c) for offshore piles can reasonably be estimated from interface shear tests.

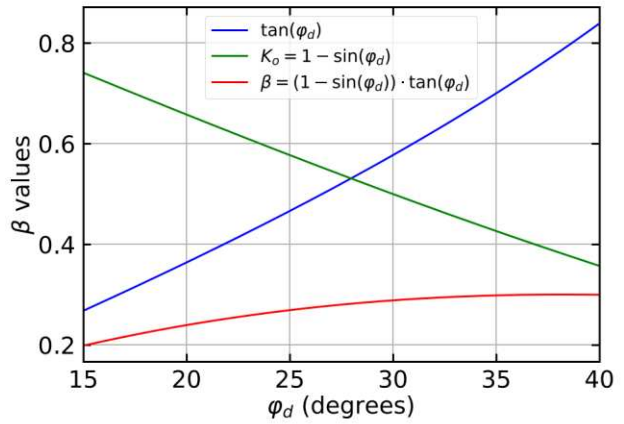

Figure 1. Comparison of β values from different equations (Burland, 1973).

The rate of loading is an important aspect of fine-grained soil-structure interfaces. Interface tests conducted by using Kaolin and interfaces with various roughness have shown that, changes in rate of loading negatively affect the interface strength of normally consolidated Kaolin (Martinez and Stutz, 2019). Similarly, interface tests on marine clay and pipe interfaces under relatively lower normal stresses have indicated an inverse relation exists between interface strength and rate of loading (Boukpeti and White, 2016).

In light of the points discussed, a comprehensive understanding of the interface strength from various aspects is important. This study examined feasibility of using rate-dependent IS-P to calculate the β values for monopiles, suction buckets, and tension pile anchors. Additionally, the impact of loading rate on the development of excess pore water pressure at the interface was studied. The measured pore water pressure values were utilized to calculate effective stress parameters which were then used to characterize the interface strength. The effective stress parameters were further used to calculate the β values for shaft resistance. The calculated β values were compared against existing data from pile load tests from previous researches.

The rate of loading not only affaect the interface strength but also the drainage conditions at the interface. Fast rate of loading can result in undrained conditions with build up of pore water pressure. Pile driving operations induce pore pressure increase in saturated soils (Rojas, E et al., 1999). Figure 2 shows the lateral earth pressures and the skin friction acting on three different types offshore wind energy turibine foundation types.

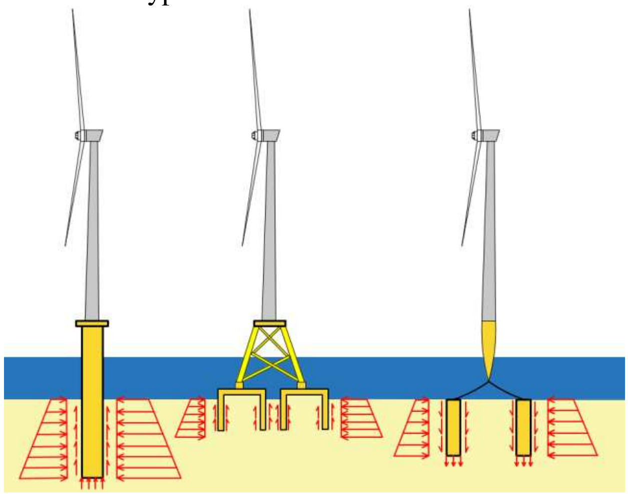

Figure 2. Earth pressure and skin-friction acting at monopile and bucket surfaces

2 MATERIALS AND METHODS

2.1 Materials

The interface shear tests were conducted on samples prepared from pure Kaolin (K) and Kaolin and Fontaine bleau NE34 sand mixtures at different ratios. The Kaolin was manufactured by Gebrüder Dorfner GmbH & Co., Kaolin und Kristallquarzsand-Werke KG in Hirschau (Germany). Some important physical and index properties of each of the two soils are given in Table 1.

Table 1 Properties of test materials

Material	Kaolin	Fontaine Bleau NE34 sand
ω_{LL}[%]	47.2	35.0
ω_{PL}[%]	-	-
PI [%]	12.2	-
d_{50}[μm	19	200
e_{max}	-	0.886
e_{min}	-	0.545
ρ_d	-	1.720
G_s	2.675	2.658

The particle size distribution for the test materials is given in Figure 3. The Kaolin (K) used in this study primarily consists of particles in the silt and clay size range, while the Fontaine Bleau (FB) sand is mainly composed of particles in the fine sand range(<0.425mm).

The samples were mixed with water contents of 1.5, 1.25, and 1.2 times the liquid limit (ωLL) for Kaolin, K60 + FB40, and K40 + FB60, respectively. The Kaolin-sand mixtures represent different material types: pure Kaolin as clayey silt, K60 + FB40 as sandy silt, and K40 + FB60 as silty sand.

Specimens were mixed using de-aired and demineralized water to minimize air entry into the mixture and to avoid free ions that could trigger chemical reactions. The paste was then consolidated in the shear box using an incremental loading sequence. The consolidation time varies, with 12 hours for K, 9 hours for K60 + FB40, and 6 hours for K40 + FB60. The sample's initial height before the application of the consolidation load was 40 mm. The attainment of full consolidation is assessed through the combined evaluation of pore water pressure dissipation and vertical displacement measurements.

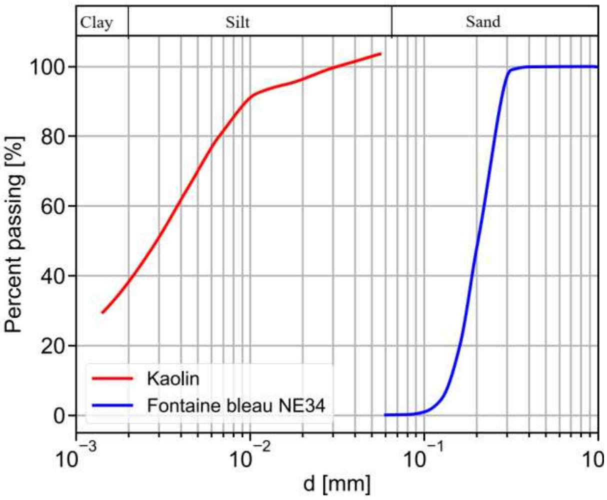

Figure 3. Particle size distribution

2.2 Methods

A novel interface shear test device equipped with three pore water pressure transducers (hereafter referred to as IS-P) was used for this study. The device is composed of two parts, a cylindrical shear box with radius, r=80 mm and height, h=40 mm and the interface. The consolidation and shearing of the samples were conducted within the same box. The test specimen was placed in the shear box and subjected to an incremental pre-consolidation stress of 75 kPa. Upon completion of the consolidation phase, the shear force was applied by the motor at a predefined shearing rate.

The vertical normal force is applied with a pneumatic cylinder while the shear force is applied by a stepper motor mounted at the side of the shear frame. The motor can apply a maximum load of 150 Ncm (Stutz et al, 2018). The sample was sheared until a shear displacement of 15mm is reached. Vertical and horizontal displacements were measure by using linear variable differential transducers (LVDTs).

The interface is fully rough surface with a pre-determined mean roughness, R_a = 107.2 µm. This is considered to be a rough surface with an interface roughness ratio, d_{50}/R_a = 1.87, for sand. R_a is described as the average deviation from the line of the mean profile height. The mean roughness for shearing surface is measured by using a digital microscope VHX-2000 from Keyence.

The IS-P tests were conducted using loading rates of 0.005 mm/min, 0.05 mm/min, 0.1 mm/min, 0.5 mm/min, 2.5 mm/min, 5 mm/min, and 10 mm/min. The drainage conditions change from drained to partially drained and then to undrained as the rate of loading increases. A velocity of 0.005 mm/min was considered as the fully drained condition for all of the specimens in this study. All tests were conducted under constant normal load (CNL) conditions. A vertical normal stress of 75 kPa at the end of the consolidation phase was applied for the shearing phase.

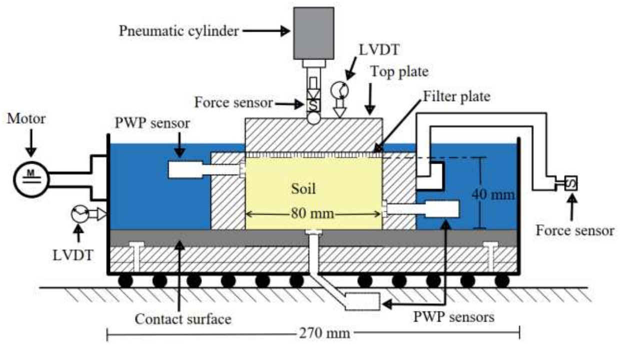

Figure 4. Interface shear test device (IS-P)

To ensure complete saturation, the samples were maintained in a water-filled bath throughout the consolidation and shearing phases. The final water content at the end of the consolidation phase for Kaolin was 0.51 for a consolidation stress of 75 kPa. After shearing, the water content of the K samples ranged between 0.30 to 0.31.

3 RESULTS AND DISCUSSION

The interface shear stress, the total normal stress, pore water pressure at the interface, vertical and horizontal displacements are measured from the tests. The rates of loading were normalized by a reference rate for fully drained conditions, v_o= 0.005 mm/min, and expressed as a dimensionless quantity [$v = v/v_o$]. Measuring pore water pressure values at the interface during shearing allowed for the calculation of the effective normal stress. The shear displacement at 15mm is considered to define the residual interface strength along with the residual interface friction angle (δ_c). The effective vertical stress at the corresponding displacement was calculated and subsequently used to determine other parameters.

For Kaolin, it was observed that the evolution of excess pore water pressure increases with increasing rate of loading until a fully undrained conditions are reached. Afterwards, with further increase in rate of loading, the magnitude of excess pore water pressure remains constant. The same results were reported from numerical studies on the effect of penetration rate piezocone tests (Silva et al., 2006). For Kaolin-sand mixtures, the pore water pressure at the interface kept on increasing with increase in rate of loading. A sudden increase in pore water pressure magnitude was recorded with further increases in loading rate.

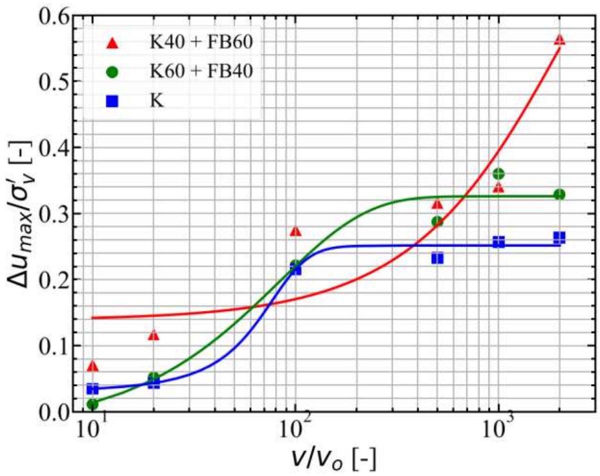

Figure 5. $\Delta u_{max}/\sigma'_v$ with respect to v/v_o

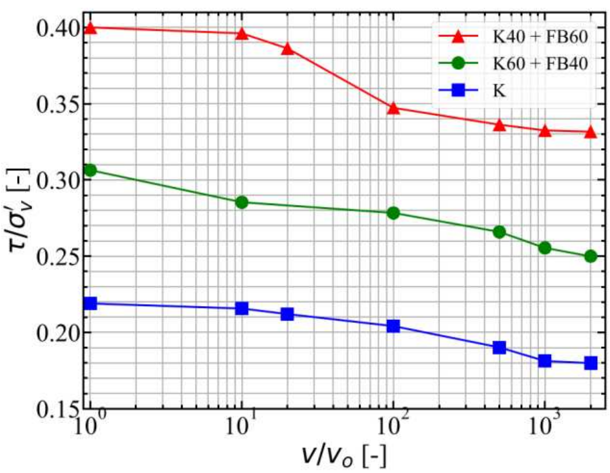

Figure 6. Interface stress ratio with respect to v/v_o

The increase in pore water pressure in sandy soils during rapid shearing occurs due to a combination of undrained conditions and the limited ability of pore water to escape during fast loading. As result, an unexpected loss of interface strength can occur. The difference in magnitude and trend of excess pore water pressure at fine-grained soil-structure interfaces, can be related to factors such as permeability of the soil. The interface stress ratio (τ/σ'_v) values for Kaolin were observed to be considerably less than those for K40 + FB60 mixtures. The values are in between 0.18 - 0.22 for Kaolin, 0.25 – 0.28 for K60 + FB40 and 0.32 - 0.4 for K40 + FB60 mixtures.

The calculated stress ratio values for Kaolin are in good agreement with the values reported by (Karlsrud et al., 2005)) for clays with nearly the same plasticity index with the Kaolin used in this study. However, the stress ratio values are slightly less than those reported from pile load tests on clay (Burland, 1973), which ranges between 0.2 - 0.3. The interface stress ratio was found to be strongly dependent on the rate of loading for both specimens as shown in Figure 6. The rate effects get diminished as the rate of loading further decreases ($\leq$ 0.001 mm/min). This could be related to the attainment of fully drained conditions. The effective normal stress at any shear displacement during shearing can be calculated ($\sigma'_v = \sigma_{v0} - \Delta u$). The β factor is found to be a function of the interface stress ratio (τ/σ'_v) and the critical interface friction angle (δ_c). For a given interface roughness, (δ_c) decreases sharply as D50 increases and remains independent of relative density (Jardine et al., 1993).

$$\tau = \sigma'_v . tan\delta \quad (1)$$

$$\delta = \arctan(\tau/\sigma'_v) \quad (2)$$

In general, the skin friction on pile surface is affected by a number of factors other than the effective horizontal stress (Silva et al., 2006).

The interface friction angle can be determined from the interface shear tests and this can be further used to calculate the shaft resistance that develops along pile- soil interfaces. Assuming the roughness of the Pile surface to be uniform in all directions, the interface shear strength can be written as shown in Equation 2.

$$\tau_f = \sigma'_h . tan\delta \quad (3)$$

$$\sigma'_h = K \cdot \sigma'_v \quad (4)$$

$$\beta = K \cdot tan\delta = \tau_f/\sigma'_v \quad (5)$$

The shaft resistance depends on the effective horizontal stress and the interface friction angle between the pile and the soil. The earth pressure coefficient K cannot be accurately determined for Pile-soil interfaces. The earth pressure coefficient at rest, $K_o = 1-\sin(\phi_d)$ for normally consolidated soil (after Jaky, 1944), was utilized to calculate the lateral earth pressure acting on the sides of the pile (Burland, 1973). For laterally loaded piles, both passive and active conditions can exist simultaneously on either side of the pile, while for cyclic loads, which are predominant in offshore structures, the conditions on opposite sides of a pile change between active and passive for each half-loading cycle. Therefore, an average earth pressure coefficient is used, as shown in Equation 6. To ensure a more conservative estimate, earth pressure coefficient at rest (K_o) is included in the average. The K values calculated in this way are relatively lower than back-calculated K values from cone penetration tests by Ganju et al., 2020).

The K values, together with the δ_c values from interface shear tests, are used to calculate the β values. The resulting β values are plotted against the normalized velocity, a dimensionless parameter used to study rate effects, as illustrated in Figure 7.

$$\mathrm{K} = \frac{K_0 + K_a + K_p}{3} \tag{6}$$

The data presented in Figure 7 is only for three different types of soils. However, the trend could change considerably if more soil types were tested.

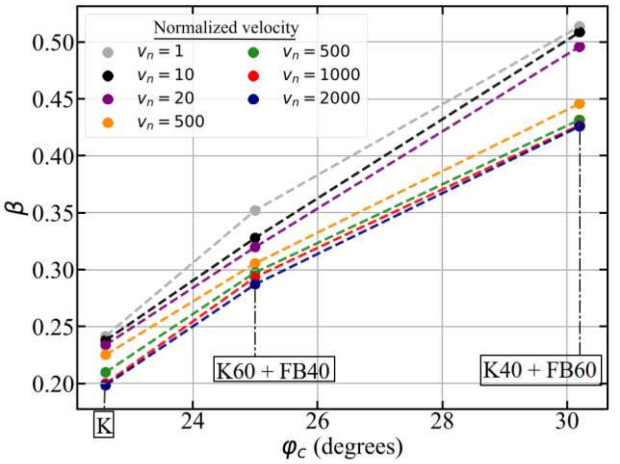

Figure 7. The β values for different types of soils and rates of loading

These is mainly because, the effect of the mean interface roughness is not linearly related to the mobilized interface strength (Martinez and Stutz, 2019). The normalized velocity is given by $v_n = v/v_o$, where v_o = 0.005 mm/min and v are the respective shearing velocity applied in each interface test.

The β values for the Kaolin specimen are in good agreement with values reported for soils with nearly the same plasticity index in NGI data base 1 (Karlsrud et al., 2005). The data reported in the NGI database 1 is solely based on the relationship between the plasticity index and the resulting β values from pile load tests as shown in Figure 8. The substantial variability in the NGI database values makes it challenging to establish a general comparison. The β values for the K60 + FB40 and K40 + FB60 mixture show a good agreement with those recommendations provided by the American Petroleum Institute (API) for sandy silts (Jegandan et al., 2012). The β value derived from these tests was found to be rate-dependent and inversely related to the normalised velocity for normally consolidated specimens.

A comparison of the test results showed strong consistency with previous studies on pile load tests (Burland, 1973; Esrig and Kirby, 1979; Karlsrud et al., 2005). The calculated β values from the IS-P tests are in the range of 0.018 and 0.40 for all rates used in this study. β values between 0.29 and 0.37 are recommended for Sandy-silts by the API whereas values between 0.2 and 0.35 for normally consolidated soils were calculated (Burland, 1973). However, β values as large as 1.53 and 1.04 were also reported (Esrig and Kirby, 1979; Burland, 1973). The significant variability in the reported results highlights the differences in site-specific conditions and the challenge of directly applying findings from other studies.

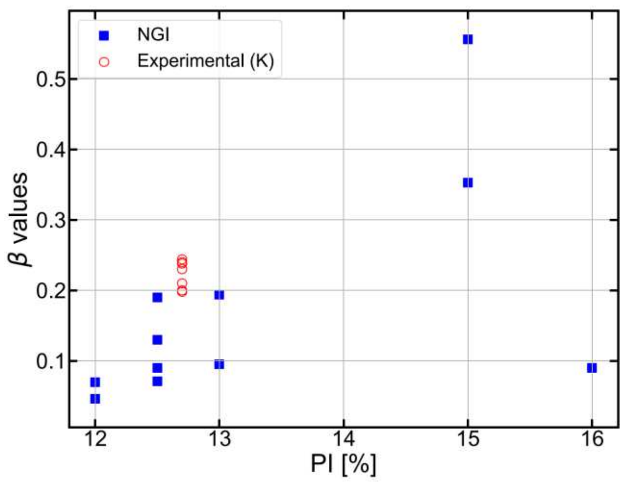

Figure 8. Comparison of β values with respect to plasticity index: Experimental results versus NGI database 1 (Karlsrud at al., 2005).

4 CONCLUSIONS

In conclusion, IS-P tests can estimate β values, enabling shaft capacity calculations for shorter monopile and suction bucket segments. The rate dependency of β values for fine grained soil-structure interfaces should be taken in to consideration. While conducting the IS-P tests, a due consideration should be given in choosing the appropriate rates of loading and mean roughness of the interface. These should be done based on the soil type, the pile type, construction technique and the specific application where the test results are intended to be used. The IS-P tests were conducted for a relatively smaller shear displacements as compared to the large deformations taking place in actual Pile-soil interfaces. However, large displacement IS-P tests could better represent the actual Pile-soil interface conditions. Apart from that constant normal stiffness tests (CNS) could better represent the in-situ conditions than the constant normal load (CNL) tests.

AUTHOR CONTRIBUTION STATEMENT

Bereket M. Gebremeskel: Conceptualization, Methodology, Data curation, Visualization, Writing-Original draft, Result Analysis.
H. H. Stutz: Conceptualization, Methodology, Writing- Review and Editing, Supervision.

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