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Passive load estimation on single piles and pile groups from visco-plastic mud-flow

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ABSTRACT: If piles are installed in creeping slopes or next to heavy loads in soft soils (with high water content) they may be loaded due to large, progressive horizontal ground displacements. In such cases pile shafts will be stressed by heavy shear forces and bending moments that may lead to serviceability problems (SLS) or even pile failure (ULS), as that soil squeezes between and around the piles. When modelling such phenomena one should consider the nonlinear material behaviour, as well as the complex nonlinear deformation and contact processes imposed by the horizontally moving ground.

The objective of this paper is to give recommendations for estimating passive loads on piles based on numerical simulations, using the visco-plastic material model of Zienkiewicz & Godbole (1974). Additionally, experimental and numerical results of a back analysis of a sophisticated large scale test for the design of a piled foundation of a runway next to a steel yard on alluvium soil in Brazil are presented. Both allow a better understanding of the state of stresses and displacements in the near-field of the piles, as well as a comprehensive understanding of soil-pile interaction; i.e. flow of soil around the pile.

1 INTRODUCION

1.1 *Literature review*

Pile foundations are selected whenever structural loads must be transferred to deeper and less compressible soil layers (e.g., gravel, rock, etc.). Such pile foundations tend to be loaded primarily in the axial direction. On the other hand, they may experience shear forces and bending moments if the pile caps are fixed to the building and are laterally acted on by horizontally moving ground.

This can occur if piles are installed in creeping slopes or next to heavy surcharge loads, especially in soft cohesive soils with high water content or in highly organic soils such as tidal mud or peat. Owing to the extreme loading conditions, the pile shafts may become heavily stressed, which in turn may lead to serviceability problems (SLS) or even pile failure (ULS), as the soil 'squeezes' between and around piles. Typical applications where pile systems/groups are loaded in the lateral direction e.g. by creeping soil have been reported by Ito et al. (1982), Poulos (1995), Ausilio et al. (2001), Cai & Ugai (2011). Laterally loaded piles associated with moving ground have also been reported for one-sided surcharge loads (Bransby & Springman 1997, Jeong et al. 2004) and piled bridge abutments (e.g. Stewart et al. 1994, Ellis & Springman 2001).

Determining the magnitude of the passive lateral loads requires not only knowledge of the material properties but also of the stress field that develops around a pile. Predicting these stresses requires taking into account the highly nonlinear stress-strain relation of the soil, as well the complex, large nonlinear deformation and contact processes between the pile and the moving ground.

Wenz (1963) was one of the first to treat the problem of passive loads on piles in soft soils in detail. Based on plasticity theory and model experiments he determined the limiting pressure on a single pile to be between $7c_u$ and $10c_u$, where c_u is the undrained shear strength of the soil. Broms (1964) proposed a limiting pressure of $9c_u$. His solution was largely empirical and had no theoretical background. Close to the ground surface this value was reduced to allow a different mode of deformation. Based on a mathematical expression of viscous clay Winter (1979) suggested that limiting pressure on a single pile is between $2c_u$ and $5c_u$. In the analysis by Randolph & Houlsby (1984), it was found that the values of ultimate soil pressure ranges roughly from $9c_u$ to $12c_u$. However, in the back-analysis of piles in unstable slopes conducted by Vigiani (1982), the ultimate soil resistance was found to be somewhat lower; i.e., between $2.8c_u$ and $4c_u$. Poulos (1995) indicated that the limiting lateral soil pressure increases linearly from $2c_u$

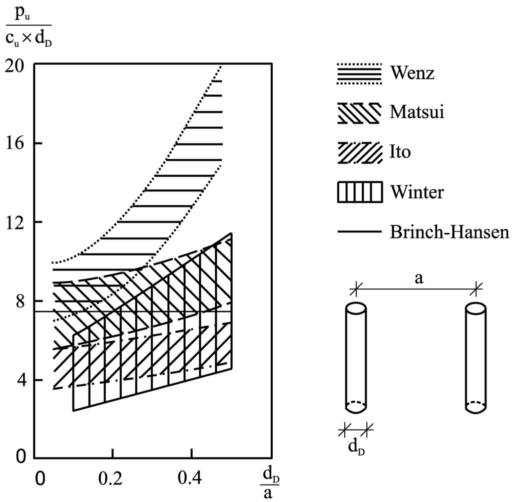

Figure 1. Passive loads p_u (line load in kN/m)on piles from horizontal ground movements, modified from Wenz (1963).

at the ground level to $9c_u$ at adepth of about 3.5 times the diameter of the pile and remained constant below that depth. Based on two-dimensional numerical analyses adopted by Bransby et al. (1999), it was found that the ultimate soil resistance was equal to $11.75c_u$ and thus slightly smaller than the value of $12.5c_u$ that is suggested by Goldscheider & Gudehus (1974). To investigate the passive load on a single pile and pile group, an analytical approach according to the 'Recommendations of the German Piling Committee – EA Pfähle' (2013) is proposed that is based on the experimental works of Wenz (1963) and Winter (1979). Figure 1 summarizes the most important approaches for different pile spacings.

1.2 *Discussion*

According to Chen (1994), the distribution of transmitted passive loads varies diversely from case to case and no general rules can be specified for practical use. Furthermore, existing analytical solutions are often limited to special boundary conditions and thus are not able to predict the soil-structure interaction correctly as required by the Eurocode EC 7-1 (EN 1997-1:2004). In recent years this has resulted in the damage of several piled bridge abutments or piled foundations of overhead bridge cranes next to heavy loads.

To overcome the shortcomings of conventional analytical approaches, many researchers have studied pile-soil interaction based on classical Lagrangian finite element method (FEM). However, this method has limitations when solving geotechnical problems with large deformations; e.g. soil flowing around a pile. Difficulties with contact algorithms and large finite element mesh distortions may lead to inaccurate results or even to a non-convergent solution of the problem. For this reason use of more sophisticated modelling techniques are being sought.

2 NUMERICAL SIMULATION

2.1 *Numerical methods for large deformations*

In recent years advanced numerical modelling techniques, such as the Arbitrary Lagrangian-Eulerian (ALE) method, the Coupled Eulerian-Lagrangian (CEL) method or the Material Point Method (MPM) have been developed to overcome numerical problems with large deformations and/or large displacements for geomechanical problems. In principle, all these methods are capable of modelling quasi static large deformation problems; e.g., the progressive creep flow of soil around a pile (Moormann & Aschrafi, 2014).

In the following sections, only small relative displacements between the pile and the soil are assumed, with the main emphasis on modelling the constitutive behaviour of the soft soil.

2.2 *Constitutive models for soft soils*

For large, progressive, deformations under plastic or visco-plastic conditions, elastic deformations assumed to be negligible with the material being assumed to flow in a viscous manner. For the purposes of this paper the flow is taken to be non-Newtonian, in which the viscosity is a function of the current equivalent strain rate.

The procedure adopted here for the material is similar to that presented by Zienkiewicz & Godbole (1974). Following Stolle et al. (2004), for plane strain conditions the state of stresses σ and the strain rate $\dot{\varepsilon}$ of an isotropic visco-plastic material can be related to one another via

$$\sigma = sI + 2\mu\dot{\varepsilon} \tag{1}$$

where $\mu =$ viscosity that ultimately depends on the failure parameters and the equivalent shear strain rate $\dot{\gamma}$, which is defined according to

$$\dot{\gamma} = \left[\left(\dot{\varepsilon}_{xx} - \dot{\varepsilon}_{yy}\right)^2 + \dot{\gamma}_{xy}{}^2\right]^{\frac{1}{2}} \tag{2}$$

In invariant form, the stress-strain relation (Eq. (1)) for the planar analysis of a viscoplastic material can be presented by

$$R = \mu\dot{\gamma} \tag{3}$$

in which R is the radius of the Mohr circle. A Mohr-Coulomb type yield criterion is adopted in this study with the yield criterion being defined as

$$F = R - \bar{c} \geq 0 \tag{4}$$

where $\bar{c}$ can be regarded as a measure of shear strength that depends on the mean stress s

$$\bar{c} = \left(c\ \cot\varphi\ - s\right)\sin\varphi = c\cos\varphi - s\sin\varphi \tag{5}$$

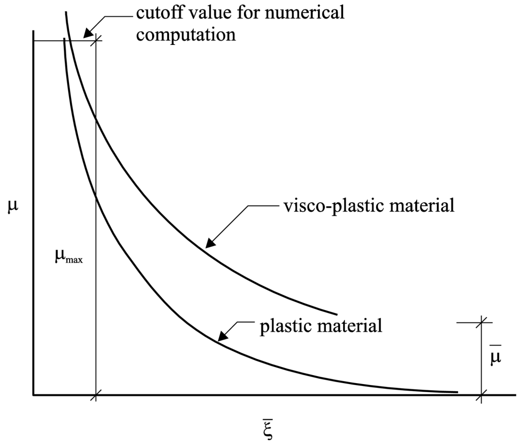

Figure 2. Viscoplastic model (Zienkiewicz & Godbole, 1974).

with c and φ being the cohesion and the friction angle of the soil, respectively. It should be noted, that flow occurs only when the yield condition of Equation (4) is satisfied. In case of a linear visco-plastic behaviour, the viscosity can be related to the strain rate and $\bar{c}$ via

$$\mu = \frac{\bar{\mu}\,\dot{\gamma} + \bar{c}}{\dot{\gamma}} \tag{6}$$

where $\overline{\mu}$ is a viscosity-type parameter, see for example, Zienkiewicz and Godbole (1974) for details. From Equation 6, one observes that the viscosity will approach positive infinity as the strain rate approaches zero (Figure 2). To avoid the numerical difficulties associated with an unbounded viscosity, a cutoff value is required. As a result, the material being modeled is allowed to deform slightly, even below yield.

3 CASE STUDY

3.1 *Benchmark: Passive load on piles*

In this section, the benchmark problem of an infinitely long cylindrical single pile, which is loaded by horizontal soil movements, is discussed.

The pile is assumed to be completely embedded in the soil and is pushed quasi statically in the horizontal direction. Since only relative displacement between soil and pile is important it is also possible to calculate the passive load on the pile, if the pile is pushed through soil.

Owing to the symmetry of the geometry and the loading conditions, only half of the problem was analysed (Fig. 3).

3.2 *Results*

For a viscosity value of 333.3 kPa-month, relative displacement between pile and soil in one month is 3 mm.

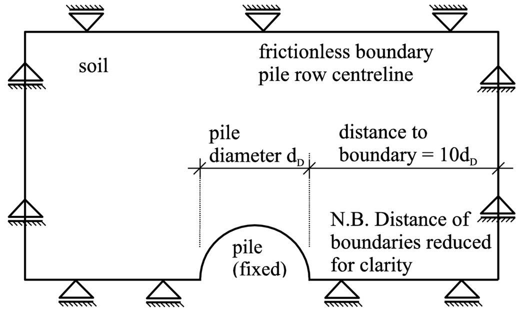

Figure 3. Boundary conditions for the 2D-benchmark problem of passive loaded piles.

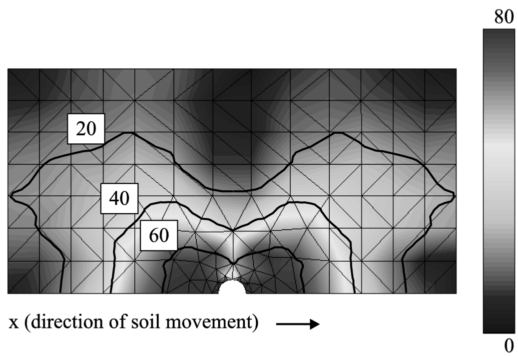

Figure 4. Equivalent shear stresses (Radius of Mohr's circle).

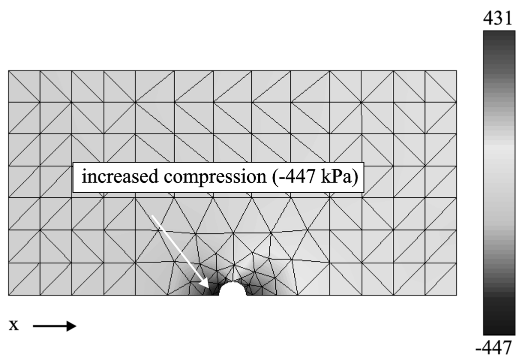

Figure 5. Horizontal stresses σ_{xx} next to the pile.

We see highly localized stress gradients in the near field of the pile (Fig. 4).

Far field pressure difference is 80 kPa. Immediately around the pile the pressure difference is ~800 kPa, as shown in Figure 5.

Increased compression on the left hand side of the pile (and decreased compression on the right hand side of the pile), as expected.

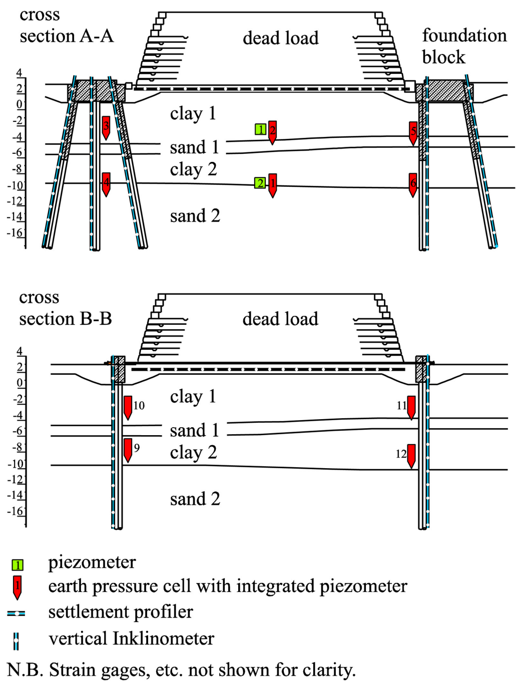

Figure 6. Large scale test for overhead bridge cranes, Brazil (for cross section A-A and B-B see Figure 7).

4 LARGE SCALE TEST

In design of piles for passive loads, in 2009 a large scale test for design of a piled foundation of a runway for a steel yard of a new steel mill on soft soil in the coastal region of Sepetiba, 60 kilometers away from the state of Rio de Janeiro in Brazil was conducted (Mühl et al. 2011). A slab yard, which has a total area of 50,000 m^2, is used to store approximately 20,000 steel slabs with regular dimension of 10.00 m × 1.25 m × 0.26 m with the weight of each slab being 25 tons. To facilitate the movement of the stored slabs, a piled overhead crane was to be constructed using steel pile profiles of Ø 813/15 mm, with a length of approximately 35 m founded in the bearing layer.

Owing to the soft soil foundation piles are loaded significantly by passive loads caused by ground displacements as anticipated. For verification of the design approach, a large scale test in the area of the highly loaded steel slabs was conducted.

For this purpose, a raft foundation with dimensions of 33 m × 40 m and a thickness of 0.6 m was built (Fig. 6 and Fig. 7). The raft foundation was loaded in 5 load steps, each followed by a consolidation phase. For each load step the load was increased by approximately 50 kN/m^2 via backfill of iron ore. At the two edges of the test field two foundations (7.0 m × 4. 0 m × 2.0 m) each with a total of 9 steel piles (Ø 813/15 mm) were installed for the overhead crane. Same of the driven piles were inclined to increase the stiffness of the system. The piles were conducted with a concrete core in the range of the upper clay layer 1. Furthermore,

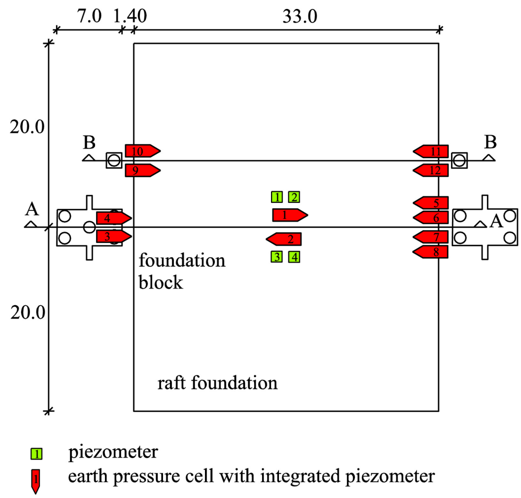

Figure 7. Top view of large scale test.

two single piles were conducted to study the load-deformation behaviour of a single pile (Fig. 6, cross section B-B).

4.1 *Ground conditions*

The ground consists of deep soft fluvial sediments of soft to very soft consistency with a total thickness up to 40 m. The water level is at the ground surface. The original swampland and grassland in the region, which is not ideal for heavy industry facility, was improved by hydraulic filling with sand before building activities. The underlying layers are sequences of clay and sand. These layers are supported by hard bedrock (mainly Granite, gneissic rock, trachyte and basalte).

Prior to the field test Standard Penetration Tests (SPT), Shear Vane Tests (VSST), Cone Penetration Tests (CPT) and Dilatometer Tests (DMT) were carried out to characterize the soil. A profile of the site is shown in Figure 6.

4.2 *Test setup and instrumentation*

An advanced measurement and monitoring system was installed prior to the field test.

During stepwise loading of the slab, changes in pore water pressure and in earth pressure within clay layers and in front of the piles were measured. Inclinometers were used to keep track of the displacement within the ground, the foundation slab and the piles. Figure 8 shows piles 1, 2 and 3 in their deformed configuration after load step 3 at 150 kPa (Mühl et al. 2011).

4.3 *Numerical model*

Owing to the symmetry of the geometry and the loading, only half of the problem was analysed. The model has the dimensions shown in Figure 9 (140 m × 70 m × 50 m). For clarity some elements are removed from the numerical model in the figure.

Approximately 200,000 8-noded hexahedron continuum-elements were used to discretize the geometry. In the immediate vicinity of the concrete plate and the 35 m long piles, the mesh was refined. After generating the initial stresses, the piles and the concrete plate were modelled. Thereafter the concrete plate was loaded stepwise similar to the field test. The piles were modelled as a linear material with Young's modulus $E_p = 2.0 \times 10^7$ kN/m^2 and Poisson's ratio $\nu_p = 0.2$. For simplification, the soil was modelled as an elastoplastic material.

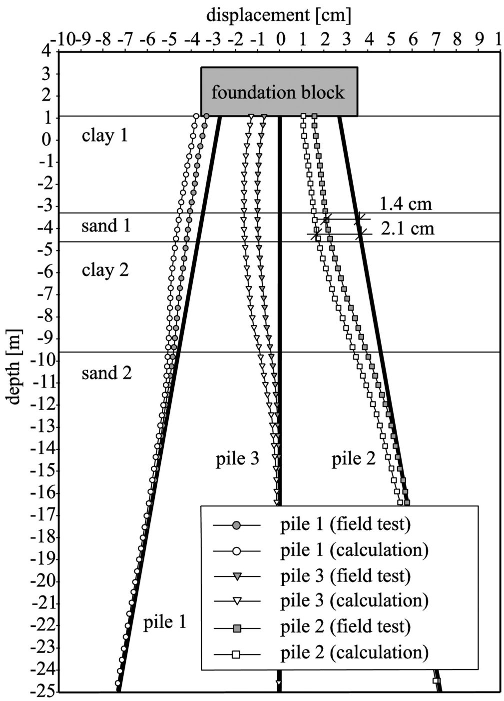

Figure 8. Pile 1, 2 and 3 in deformed configuration after load step 3 at 150 kPa (Mühl et al. 2011).

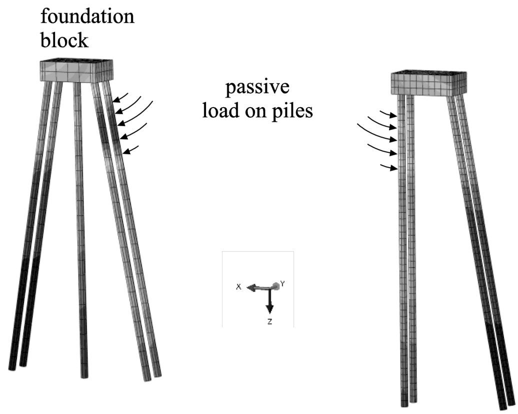

Figure 10. Total displacements of the foundation piles.

Table 1. Comparison of field measurements and FEM-results.

Load step	Settlement u_{max} (boundary raft foundation) [mm] measurement	FEM	Horizontal displacement w (pile 2) in sand layer 1 [cm] measurement	FEM
2 (107 kN/m^2)	45.7	57.3	8	13
3 (154 kN/m^2)	62.1	75.9	14	21
4 (202 kN/m^2)	90.6	130.0	26	36

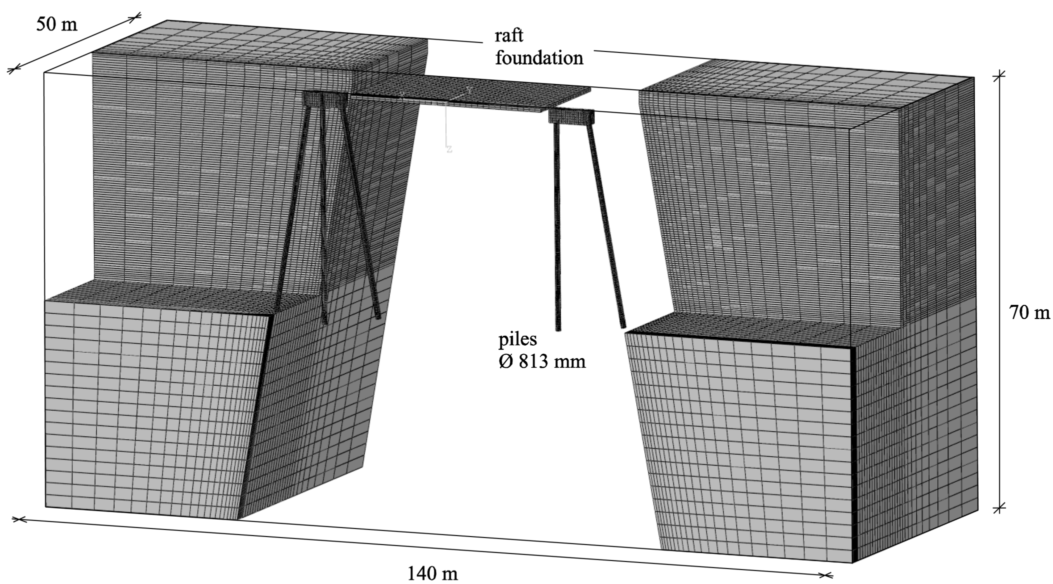

Figure 9. Numerical model for back-analysis of large field test in Brazil (for clarity some elements are removed).

The contact between piles and soil is modelled according to the master-slave concept. The tangential shear-behaviour is modelled with an elastoplastic Mohr-Coulomb constitutive model.

4.4 *Results*

For the back-analysis fundamental simplifications were made. Nevertheless,the calculations showed a good correlation to the deformation behavior that was observed during the field test. Figure 10 shows the total displacements of the foundation piles after load step 3.

The maximal horizontal pile deformation appears approximately 6 to 7 m below the ground level in the clay layer 1 and 2. Numerically determined pile deformations are on average 20% larger than the measured ones (Tab. 1). The calculated deformations of the foundation slab (boundary of slab) shows the same behavior. A comparison is also shown in Table 1.

5 CONCLUSIONS AND OUTLOOK

An important application in the geotechnical field has been tackled, i.e. the flow of soil around a pile The benchmark test examines the state of stresses and displacements in soil as well as in piles and provides a comprehensive understanding of complex soil-pile interaction in soft soils associated with creep. For the adequate numerical modelling of large displacements (e.g. dowels in creeping ground) further calibration of model parameters is needed and advanced modelling techniques are required.

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