Numerical Investigation of Passive Loads on Piles in Soft Soils

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ABSTRACT: If piles are installed in slopes or next to heavy loads, especially in soft cohesive soils, piles are often loaded by horizontal ground displacements. In this case the pile shafts are stressed by shear forces and bending moments. Initially, a structured overview of existing analytical methods for estimating passive loading on piles by horizontal movement of surrounding soil is presented. While one component of passive load is determined by the shear strength of the soil, the other often negligible component depends on the relative velocity between the pile and surrounding soil. For mobilizing the maximum passive load on pile, the soil often has to undergo large deformations. In classical Lagrangian Finite Element Method these deformations distort the finite element mesh which might lead to inaccurate results or the numerical analysis does not converge to a stable solution. Hence, numerical study of pile soil interaction needs more sophisticated modelling techniques like the Arbitrary Lagrangian-Eulerian (ALE) method, the Coupled Eulerian-Lagrangian (CEL) method or the Material Point Method (MPM) to minimize numerical problems caused by distorted mesh. These methods are discussed briefly and their potential is shown for a simple 2D plane strain problem of a single passive loaded pile.

1. INTRODUCTION

If piles are installed in slopes or next to heavy loads, especially in soft cohesive soils with high water content and soft or even more unfavourable consistency or highly organic soils e.g. tidal mud or peat, piles are often loaded by horizontal ground displacements. In such cases pile shafts will be stressed by heavy shear forces and bending moments, which may lead to serviceability problems (SLS) or even pile failure (ULS), when soil ´squeezes´ between and around piles. In literature these piles are called as passive loaded piles (Bransby, 1996). Typical examples are slopes with pile systems (e.g. dowels), as shown in Figure 1 (e.g. Ito et al., 1982; Gudehus & Schwarz, 1984; Schwarz, 1987; Poulos, 1995; Ausilio et al., 2001; Cai & Ugai, 2011). But also in case of heavy one sided loads (e.g. Oteo, 1977; Bransby & Springman, 1997; Jeong et al., 2004), piled bridge abutments (e.g. Stewart et al., 1994; Ellis & Springman, 2001) and excavation works, piles might get, in particular in soft soils, heavy more or less unexpected passive loads. Figure 1 shows the most important design situations for passive loaded piles. The negative effect of passive loads on piled foundations was shown in many damages e.g. of piled bridge abutments or piled foundations of overhead bridge cranes next to heavy loads.

Piled foundations of overhead bridge cranes for stock yards on soft soils represent particular critical design situations. One case in 1960s lead in Bremerhaven (Germany) to unexpected heavy horizontal ground displacements and consequently to overstressed foundation piles causing shift distortions of the crane runway (Leussink & Wenz, 1969). For investigation of the process amongst others an instrumented 6 m long steel pile was installed and the behaviour of the instrumented pile was monitored. Three years after applying a load next to this testing pile, passive loads caused a burst of the testing pile. The pile deflection was approximately 1.0 meter and ground was already flowing around the pile.

Because there is still some lack of knowledge in design of piles for passive loads, in 2009 a large scale test for design of a piled foundation of a runway on soft soil for a steel yard in Brazil was conducted (Mühl et al., 2011). Numerical results of the large scale test are reported e.g. by Aschrafi et al. (2013).

2. LITERATURE REVIEW

According to Eurocode EC 7-1 (EN 1997-1:2009) the design of passive loaded piles can be based on analytical, semi-empirical or numerical solution. Thereby a classification in geotechnical category 3 according to EC 7-1, i.e. the most difficult category, is requested. For design of piles, ground displacements can be treated both as action followed by an interaction analysis to determine forces, lateral displacements and strains in the pile or as an upper bound to force, which the ground can exert to the pile. Evaluation of this force shall take account of the strength of the soil and the source of the load, represented by the weight or compression of the moving soil or the magnitude of disturbing actions (EN 1997-1:2009).

In more complex situations and in accordance to EC 7-1 passive loads have to be determined with numerical methods, considering the interaction between piles and the moving ground mass. Comparison of numerical results with existing analytical solutions is recommended to the engineer.

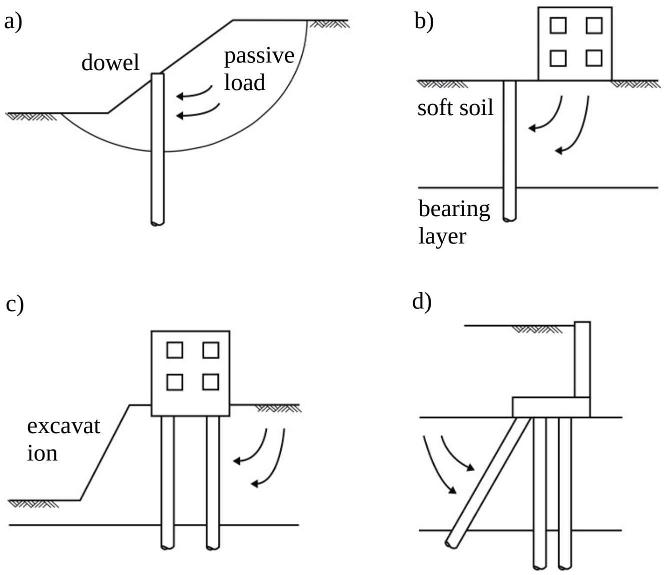


Figure 1 Design situations for passive loaded piles: a) piled slopes (dowels), b) heavy surcharge loads, c) excavations, d) piled bridge abutments with inclined piles

2.1 Analytical methods

The quantity of the passive load depends on the magnitude and velocity of the ground movement as well as on the pile stiffness and the geometrical boundary conditions such as center to center distance of the piles and thickness of the soft soil layer. According to the bending stiffness, roughness of pile surface and the quantity of ground displacement, two cases have to be considered, when designing piled foundations for passive loads: 1. case: Piles are able to resist passive loads regardless of their failure and are only deformed according to their bending stiffness and the magnitude of horizontal ground displacement. If pile displacement is less than the ground displacement, soil will flow around piles. The quantity of the load depends on the velocity of soil. 2. case: The piles do not have a certain bending stiffness to resist passive loads, consequently piles are partially deformed like the ground. Hereby failure might occur, if pile deformation exceeds the maximum allowable deformation of the piles.

Fundamental features of passive loaded piles show similarity to flow around a cylinder in a viscous fluid (Figure 2). Ito and Matsui (1975, 1982) proposed a theoretical method, called theory of plastic flow, where the lateral force on a pile is calculated by considering the soil as a visco-plastic medium. A similar theory was presented by Winter (1979) based on viscous properties of cohesive soils, which involved an analytical solution for the passive load on a pile in a viscous soil. Firat et al. (2005) investigated the flow of a visco-plastic soil around a pile in a row. He performed calculations with variation of fundamental key parameters.

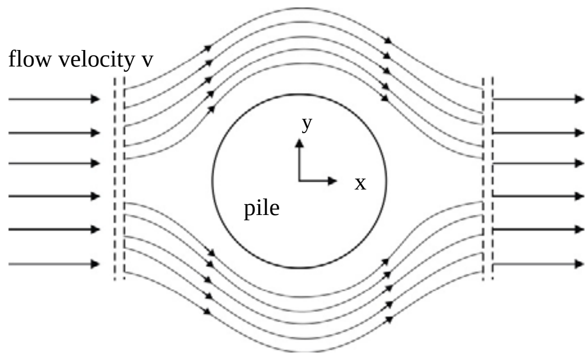


Figure 2 Flow of visco-plastic soil around a single pile (Firat et al., 2005)

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´ (EA-Pfähle 2013, 2nd edition) is proposed, which is mainly based on the experimental works of Wenz (1963) and Winter (1979). The necessity of designing piles for passive loads can be estimated by a global failure analysis according to DIN 4084, adopting the partial safety factors from Eurocode EC 7-1 (EN 1997-1:2004). Determination of the passive load is done by a limit value determination for loads resulting from ground movements lateral to the pile axis. Decisive is the smaller total passive load on the piles calculated from the characteristic resulting earth pressure $\Delta p_{e,k}$ and the characteristic flow pressure $p_{u,k}$ whereby the effect resulting from flow pressure $P_{u,k}$ and the resulting earth pressure ΔE_k are determined over the entire height of the action.

In case of determination of the resulting earth pressure, it is assumed that the piles support the soft soil with their bending stiffness, whereby for the flow theory it is assumed that the shear strength of ground is fully mobilized and the plastified ground flows around the pile. In the latter case, the pressure $p_{u,k}$ on the pile can be determined independent of depth according to EA-Pfähle (2013) with Eq. (1):

$$p_{u,k} = 7 \cdot \eta_a \cdot c_{u,k} \cdot a_s \tag{1}$$

thereby η_a is a calibration factor for the spatial pile arrangement of a single pile in a pile group (factor 1 to 5) according to Wenz (1963) and c_u the undrained cohesion of the soil. In case of a square cross-section a_s is the pile width normal to the direction of flow, apart from that a_s is the diameter of a round pile.

In literature the approach for passive loads on piles varies because of constitutive and kinematic assumptions. Passive loads on piles based on analytical methods are given in a very wide range of

$$p_{u,k} = 3 \sim 12.5 \cdot c_{u,k} \cdot d_D \tag{2}$$

at which d_D is the diameter of the pile.

In last decades, analytical and semi-empirical approaches for reliable determination of passive loads on piles have not been further developed. In Table 1 most important analytical and empirical approaches are summarised from an extensive literature review.

2.2 Numerical methods

In recent years, Finite Element Method (FEM) has been considered the main tool for solving geotechnical problems. FEM is often preferred for their speed and simplicity. Fundamental research for pile-soil interaction for passive loaded piles, based on finite element analyses, is shown by many researchers (e.g. Bransby, 1996; Bransby & Springman, 1999; Xu & Poulos, 2001; Miao et al., 2006; Georgiadis et al., 2013). Depending on the stiffness in the elastic region, deformation of approximately 2% of the pile diameter are necessary to mobilize maximum passive load on a pile in fully undrained, elastic-plastic soil (Bransby & Springman, 1999).

The problem of considering a gap behind the pile was studied by Bransby & Springman (1996). They mentioned that the occurrence of a gap is more significant for piles at low stresses (e.g. dowels) compared to piles with high normal stresses around the piles (e.g. caused by surcharge loads or great depth). For the second case it has been observed that no gapping occurs.

Several researchers studied also application-orientated problems of passive loaded piles such as piled bridge abutments (Stewart et al., 1994; Ellis & Springman, 2001; Kelesoglu & Springman, 2011), piles adjacent to heavy one sided surcharge loads (Bransby & Springman, 1996) and piled slopes (e.g. Chaoui et al., 1994; Cai & Ugai, 2000; Pan et al., 2002; Chen & Martin, 2002; Ellis et al., 2010; Kanagasabai et al., 2011) using small strain deformation theory. Based on numerical discretisation of the contact zone between pile and soil and constitutive law only less relative displacements could be modelled with the classical small strain Finite Element formulation.

Studying geotechnical problems like collapse of a pile construction e.g. dowels in a slope undergoing large ground displacements or installation effects of piles or spud cans in offshore engineering, it is evident that large deformation can occur. Fundamental research in modelling large deformations has been done by e.g. Qui et al., 2010; Henke et al., 2010 and Kafaji, 2013. Tian et al. (2011) studied large-amplitude penetration of a full flow penetrometers and pipeline using large deformation analyses.

3. DISCUSSION ON LITERATURE REVIEW

In many geotechnical applications the material undergoes large deformations e.g. dowels in moving ground. Existing analytical methods are often limited to special boundary conditions respective are not able to predict the soil-structure interaction in a reliable way for design of geotechnical constructions. Because of this reasons, in recent decades finite element methods became a powerful tool for a wide range of geotechnical applications.

It is evident that classical finite element method has many shortcomings solving geotechnical problems with large deformations. Especially contact problems and large finite element mesh distortions may lead to inaccurate results or even to a convergent solution of the problem. There is still no understanding in the soil-pile interaction and failure mechanism of the soil under large deformation conditions. Because of this reason it is necessary to use more sophisticated modelling techniques like the Arbitrary Lagrangian-Eulerian (ALE) Method, the Coupled Eulerian-Lagrangian (CEL) Method or the Material Point Method (MPM).

Table 1 Analytical methods for estimating passive loads on piles in soft cohesive soils

Author	Formulation	Comment
Brinch-Hansen & Lundgren (1960)	$p_u = 7.5 \cdot c_u \cdot d_D$	Load from base failure of a deep based vertical foundation.
Wenz (1963) [(1)]	$\frac{p_u}{c_u \cdot d_D} = \chi \cdot \psi \cdot (2 + \xi \cdot \pi)$	Based on plasticity theory and model experiments.
Broms (1964)	$p_u = 9 \cdot c_u \cdot d_D$	Empirical formula without theoretical background. Close to the ground surface the load was reduced to allow for different mode of deformation.
Smoltczyk (1966)	$p_u = 4.4 \cdot c_u \cdot d_D$	Only shear stresses acting on surface of the pile.
Goldscheider & Gudehus (1974)	$p_u \leq 12.5 \cdot c_u \cdot d_D$	Assumption of a kinematic failure mechanism (sickle).
Ito & Matsui (1975) [(2)]	$p_u = c_u \cdot \left[D_1 \cdot \left(3 \cdot log\frac{D_1}{D_2} + \frac{D_1 - D_2}{D_2} \cdot \right.\right.$ P = · $\left.\left. tan\frac{\pi}{8}\right) - 2 \cdot (D_1 - D_2)\right] +$ $P = + \gamma \cdot h \cdot (D_1 - D_2)$	Development of two different approaches for pile groups: 1) Theory of plastic deformation (see formula) and 2) Theory of plastic flow (Ito & Matsui, 1975).
Winter (1979) [(3)]	$\frac{p_u}{c_u \cdot d_D} = k_0 \cdot \left[1 + I_{v\alpha} \cdot \left(\frac{\frac{v_0}{\dot{\varepsilon}_\alpha}}{a - d_D}\right)\right]$	Based on a mathematical expression of a viscous clay.
Randolph & Houlsby (1984) [(4)]	$\frac{p_u}{c_u \cdot d_D} = \pi + 2\Delta + 2 \cdot cos\Delta +$ $\frac{p_f}{c_u \cdot d_D} \quad + 4 \cdot \left[cos\left(\frac{\Delta}{2}\right) + sin\left(\frac{\Delta}{2}\right)\right]$	Lower bound analytical expression for a single pile with different roughness according to classical plasticity theory.

(1): χ = factor for the shape of the pile; ψ = factor for pile arrangement; ξ = factor for embedment of the pile
(2): D_1 = center distance; D_2 = inside width between the piles; h = thickness; c_u = undrained cohesion; γ = unit weight
(3): k_0 = shape parameter; $I_{v\alpha}$ = viscosity index; v_0 = flow rate; $\dot{\varepsilon}_\alpha$ = shear rate;
a = center distance normal to the direction of flow
(4): Δ = factor for roughness of the pile surface

4. NUMERICAL METHODS FOR MODELLING LARGE DEFORMATIONS

Since first application in the middle of the last century, computational methods have become a very powerful tool in geotechnical engineering for analysing problems like deep foundations, excavations and tunnels etc. with increasing complexity. By trying to make accurate predictions e.g. soil flowing around a pile with standard numerical methods, the limits of conventional numerical simulations are reached.

Determining the magnitude of the passive load requires not only knowledge of material properties, but also of the developing stress field surrounding the pile. Predicting these stresses requires taking into account the highly nonlinear stress-strain relationship of soil, as well as the complex nonlinear deformation and contact processes imposed by the horizontal moving ground around a pile.

This article deals with the proper reproduction of large deformations by numerical simulation of passive loaded piles. Here, the focus lies on quasi-static problems where inertia effects can be neglected.

4.1 Finite Element methods

The Finite Element Method (FEM) is the most important numerical method in computational geomechanic applications. The theory of traditional FEM is suitable for geotechnical problems, when element distortions are moderate. Theory can be found in e.g. Zienkiewicz & Taylor (1967) or Bathe (1996). The interface is precisely defined and tracked and the Lagrangian elements contain the same material throughout the calculation and the material moves only with the deformation of the mesh. Soil-structure interaction is generally modelled with discrete interface elements. The presently used FE-programs provide often a calculation procedure for geometric nonlinearity, which is based on an **updated Lagrange (UL)** Finite Element formulation. In FE-analysis with updated Lagrange procedure, the stiffness matrix is updated due to the new geometrical positions of the deformed elements. In addition to that, a special definition of stress rate is adopted that includes rotational terms. For geometric nonlinearity, however, discrete interface elements cannot be used in an unrestricted way. For this reason, kinematic contact algorithms according to the master-slave principle often tend to be applied (Moormann & Katzenbach, 2002).

Nevertheless, in many nonlinear geotechnical applications the material undergoes large deformations and even the updated Lagrangian Finite Element Method is not suitable anymore. These deformations distort the finite element mesh and the nonlinear boundary conditions. For this reason, the distorted mesh is unable to provide accurate results and can slow down the convergence of the solution or even the numerical analysis does not converge to a stable solution. Figure 3 shows the mesh in deformed configuration after applying a prescribed displacement of $0.5d_D$ to the rigid pile with rough surface. It is obvious that for this distorted mesh, but also with a finer mesh discretisation and different pile-soil interaction properties, one cannot trust in numerical results anymore. Thus, for excessive mesh distortions or material brakes up, different numerical formulations should be used, in order not to lose accuracy in the solution.

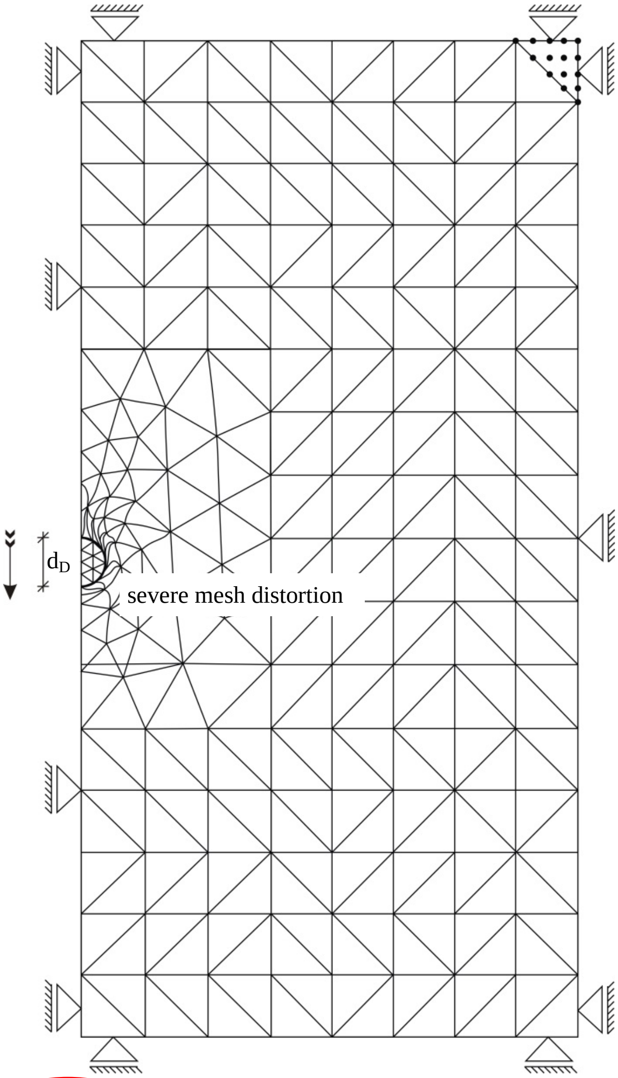


Figure 1 Finite Element mesh around a pile in deformed configuration, coarse mesh (UL-FEM)

In recent years some Finite Element methods, based on the Arbitrary Lagrangian-Eulerian (ALE) method or Coupled Eulerian-Lagrangian (CEL) method have been developed to overcome numerical problems by the distorted mesh. In case of pure Lagrangian description (particle description) the movement of the continuum is specified as a function of the material coordinates and the time. The nodes of the Lagrangian mesh move together with the material. Therefore, the interface between two parts is precisely tracked and defined. In Eulerian description (field description) the movement of the continuum is specified by a function of the spatial coordinate and time. An Eulerian reference mesh which remains undistorted is needed to trace the free motion of the material in the Eulerian domain. In an Eulerian description no element distortions occur, but numerical diffusion can happen in case of two or more materials in the Eulerian domain.

The **Arbitrary Lagrangian-Eulerian (ALE)** method is an effective method, to analyse large deformation problems. This method was first introduced by Hirt et al. (1974) to solve fluid dynamic problems or later metal forming problems (e.g. Khoei et al., 2008). This method uses a finite mesh with nodes, which may move with the material (Lagrangian description) or be held fixed in space (Eulerian description). The mesh is partially attached on material and can become independent where necessary. Initially, Lagrangian body and Eulerian body entirely overlap each other and have the same mesh discretisation. Mesh motion (Lagrangian) is constrained to the material motion (Eulerian) only where necessary (at free boundary). Otherwise, material motion and mesh motion are independent. The ALE method can overcome the mesh distortion while representing the boundary conditions correctly.

According to Hirt et al. (1974) one time step of an ALE-calculation is based on three phases, incipient with a classical explicit Lagrangian calculation phase. After this phase, the momentum equation is identical to a time-step in a standard Lagrangian analysis. The following optional adaptive meshing phase respectively smoothening phase moves only nodes independently from material to reduce mesh distortion. Nodes, which are specified to adaptive domains, are frequently adapted to keep reasonable element shape during large deformation analyses. However, the topology i.e. the number of elements and connectivity of each element is not altered. Adaptive meshing could also be used to analyse pure Lagrangian problems as well as pure Eulerian problems. Adaptive meshing techniques have been proposed by e.g. Benson (1989). In a final Eulerian phase the solution obtained from the Lagrangian phase is remapped onto the new relocated mesh (advection phase), which was developed through the adaptive meshing algorithm. Examples with the ALE method can be found in e.g. Tian et al. (2011).

Origin, the **Coupled Eulerian-Lagrangian (CEL)** method was developed by Noh (1964) and further developed e.g. by Benson (1992) and Benson & Okazawa (2004). This method, which attempts to capture the strength of the Lagrangian and the Eulerian method, is implemented in the commercial code ABAQUS/Explicit. For geotechnical problems, a Lagrangian mesh is used to discretize structures; while an Eulerian mesh is used to discretize the subsoil. The Eulerian material is tracked as it flows through the Eulerian mesh computing its Eulerian volume fraction (EVF). If an Eulerian element is completely filled with material, its EVF is 1. If there is no material in the element, its EVF is 0 (Dassault Systéms, 2013).

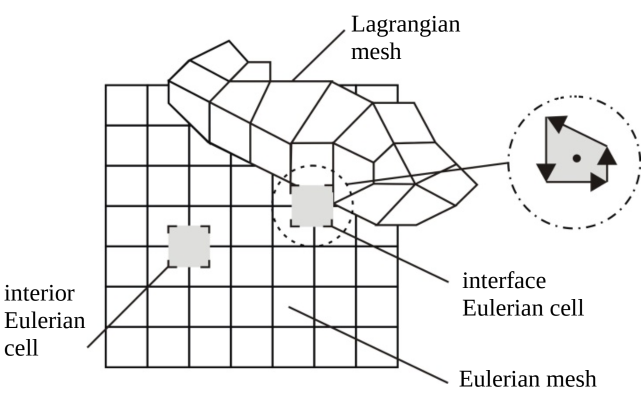


Figure 4 Principle representation of Noh´s Coupled Eulerian-Lagrangian method (Qui, 2012)

The interface between structure and subsoil could be represented using the boundary of the Lagrangian domain. On the other hand, the Eulerian mesh, which represents the soil that may experience large deformations, has no problems regarding mesh and element distortions. According to Noh (1964) initially the pressure on the interface of the Eulerian interface cells are integrated to calculate the force acting on the Lagrangian nodes at the interface. Following, the motion of the Lagrangian domain is calculated and the portion of the domain at the time $t = t^{n+1}$ is determined using the new position of the Lagrangian domain. Finally, the discretized Eulerian equations are solved to obtain the new pressure.

In Abaqus contact between Eulerian materials and Lagrangian materials is enforced using a general contact algorithm that is based on a penalty contact method (Dassault Systéms, 2013). This contact algorithm does not enforce contact between the Lagrangian elements and the Eulerian elements, i.e. Lagrangian elements can move through the Eulerian mesh without resistance until they encounter an Eulerian element filled with material (EVF ≠ 0). The implemented penalty contact algorithm creates seeds on the Lagrangian element surfaces, while anchor points are created on the Eulerian material surface. The penalty method approximates hard pressure overclosure behaviour. Small penetration of the Eulerian material into the Lagrangian domain is allowed. The contact force F_P between seeds and anchor points is proportional to the penetration distance d_p

$$F_P = k_p \cdot d_p \tag{3}$$

where the factor k_p is the penalty stiffness which depends on the Lagrangian and Eulerian material properties. This method is less stringent compared to the classical kinematic contact method (master-slave concept).

An illustration of the penalty contact algorithm is shown in Figure 5.

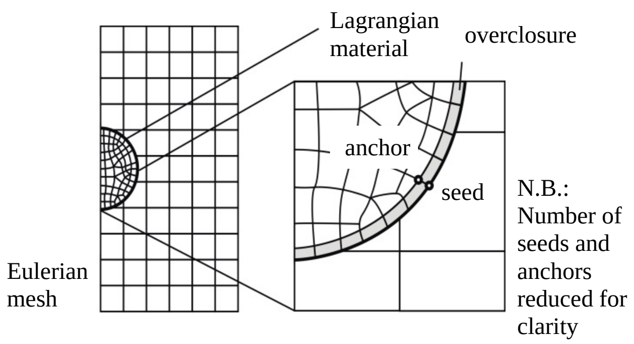


Figure 5 Schematic illustration of the contact between Lagrangian part (pile) and Eulerian part (soil)

Qui et al. (2010) or Henke et al. (2010) showed the potential of the CEL method for geotechnical applications undergoing large deformations.

Both CEL and ALE methods use an explicit time integration scheme. Thereby, the unknown solution for the next time step can be found directly from the solution of the previous time step. In this case no iteration procedure is needed. However, explicit calculations are not stringently stable, which means, that numerical stability has to be guaranteed by introduction of a critical time step Δt_{crit}. The step size depends on the characteristic element length L_e and the dilational wave speed c_d (Bathe, 1996).

Both methods (ALE and CEL) combine the benefit of a pure Lagrangian and pure Eulerian description of the movement of a continuum in time. Simplified mesh structures of a passive loaded pile with the ALE and CEL method are shown in Figure 6.

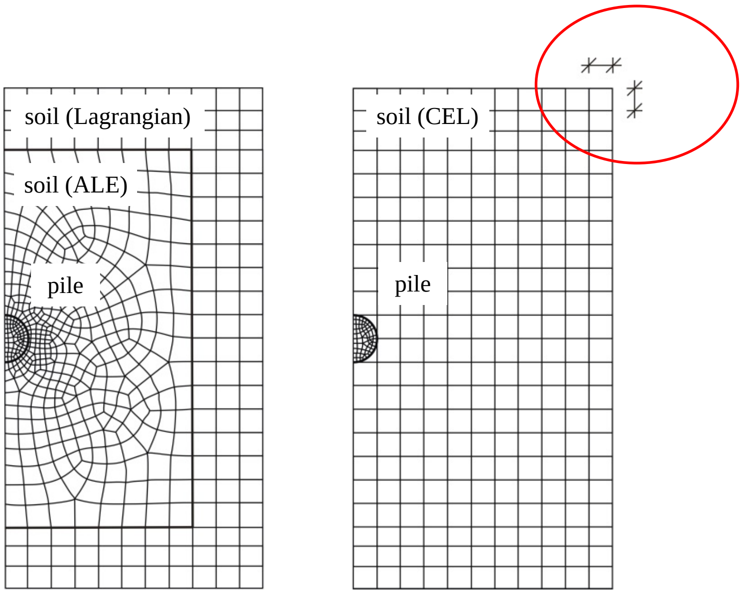


Figure 6 Simplified mesh structure for ALE model (left) and CEL model (right)

4.2 Meshbased particle methods

The **Material Point Method (MPM)** is a mesh based particle method and has been developed to overcome the difficulties of the FEM. The method is based on a finite element mesh and a cloud of points, called material points that move through a fixed Eulerian grid. These material points represent subregions of a soil material.

Deformation of the solid is represented by the movements of Lagrangian material points (particles). The material points carry all physical properties of the continuum such as mass, momentum, material parameters, strains, stresses as well as external loads. The computational Eulerian mesh and its Gauss points carry no permanent information. The computational grid is used to determine incremental displacements by solving the stiffness equations as in standard Finite Element Method. The schematic representation of a classical calculation cycle of the Material Point Method contains the following steps (Figure 8): At the beginning of a time step, information is transferred from particles to the computational background mesh. This includes amongst others external load information. The element stiffness matrices are constructed and assembled into a global stiffness matrix. After determining the incremental solution of the field equations as in Lagrangian way, the solution is mapped from the computational mesh back to the material points to update their informations (convective phase) respectively the location and stresses of the material points. After updating information, mesh is reset to its initial configuration.

In recent years the Material Point Method has been applied successfully to dynamic and quasi-static large deformation problems (see e.g. Beuth, 2012 or Kafaji, 2013).

4.3 Benchmark: Passive load on single pile

In this part, the benchmark problem of an infinitely long cylindrical single pile which is loaded by purely horizontal soil movements is discussed. Undrained loading of passive piles is analysed using UL-method, ALE-method, CEL-method and the Material Point Method.

In Figure 7 a vertical section through the half space is shown. The pile is completely embedded into soil and pushed in horizontal direction. To simplify calculations, soil is modelled as perfectly plastic cohesive material and is assumed to be weightless. Since only relative displacement between soil and pile is important, it is also possible to calculate the passive load on the pile, if pile is pushed into soil.

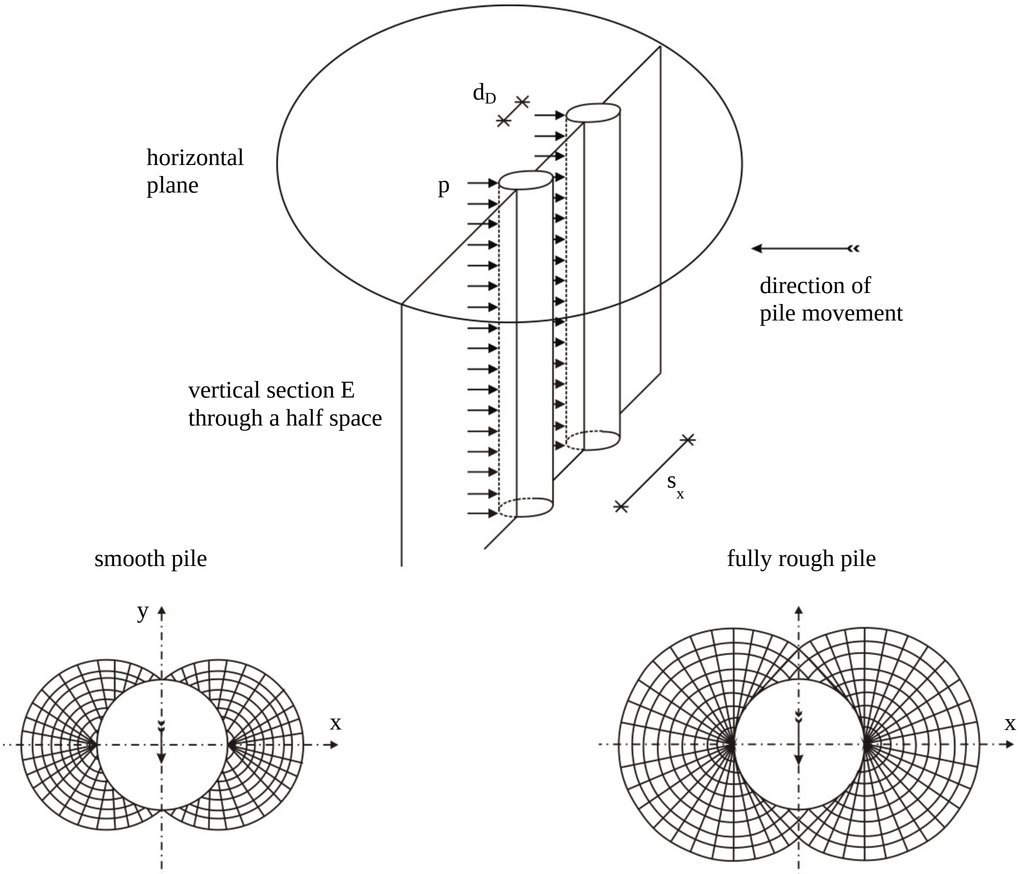


Figure 7 Assumptions of flow around a pile and slip lines of deforming zones; extreme cases smooth pile and fully rough pile (Randolph & Houlsby, 1984)

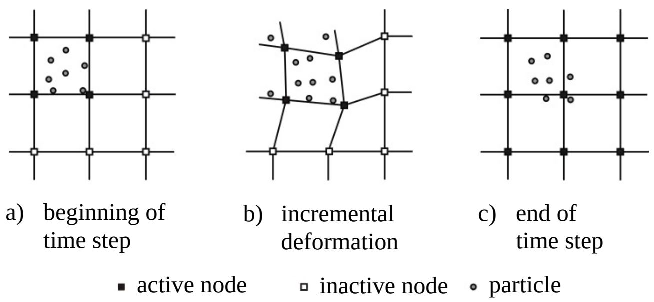


Figure 8 Basic concept of MPM (Kafaji, 2013)

The magnitude of force per length P_u, which is needed to push the pile into soil, could be expressed by the solution after Randoph & Houlsby (1984). The force P_u of a single pile ranges from $(6+\pi)\ c_u \cdot d_D = 9.14\ c_u \cdot d_D$ for a smooth pile up to $(4 \cdot \sqrt{2} + 2\pi)\ c_u \cdot d_D = 11.94\ c_u \cdot d_D$ for a perfectly rough pile per meter length.

4.3.1 Geometry and discretization

Due to the symmetry of the geometry and the loading conditions, only half of the problem was analysed. The problem was reduced to a plane strain problem in plasticity theory, in which the passive load is calculated on a long cylinder, which is moved laterally through an infinite medium. Pile diameter was taken as 1.0 m. Pile loading was subsequently simulated by applying prescribed displacement y to all nodes on the pile diameter. The geometrical boundary conditions and load definitions are summarised in Figure 9. Soil is fully attached to the pile (no gap). Boundaries were placed at a distance of $25d_D$ from the pile and fixed in normal direction.

Mesh size of the elements for UL-method, ALE-method, CEL-method and MPM was studied before calculation. Basic assumptions for ALE method and CEL method are shown in Figure 6.

In dynamic calculations, special attention should be paid to reflecting waves from model boundaries with e.g. infinite boundary elements or special boundary conditions with damping properties. For the introduced benchmark problem of a passive loaded single pile effects from reflecting waves have not been considered because of quasi static loading conditions.

Contact was modelled between the piles and the soil. Both smooth pile and fully rough pile (and medium rough) conditions are investigated.

4.3.2 Constitutive law

The piles were modelled as linear material with the elastic properties of reinforced concrete (Young's modulus $E_p = 2.0 \times 10^7$ kN/m^2 and Poisson's ratio $\nu_p = 0{,}2$, while the soil was modelled as a Tresca material with undrained cohesion $c_u = 100$ kN/m^2, undrained Young's modulus $E_u = 300c_u$ and undrained Poisson's ratio $\nu_u = 0.495$. Various adhesion factors α (= τ_f/c_u, where τ_f is the limiting pile-soil adhesion) and pile spacings s_x were considered, while the pile diameter was taken as $d_D = 1.0$ m in all analyses.

According to the Tresca failure criterion, which was adopted for the soil around the pile, the shear strength is independent of the stress level in soil. Consequently, FEM results are independent of the initial stresses. Therefore stresses were specified as zero at the beginning of each analysis.

In case of a rate-independent constitutive model, the velocity of pile movement respectively soil movement has limited influence on the numerical results, which means, that the computational time of an explicit analysis can be shorten by an increase of the velocity.

According to Bransby and Springman (1999) assumptions of fully undrained soil behaviour, plane strain conditions and no gapping produce worst case results for passive loading. In their opinion, these conditions are often fulfilled in real site conditions.

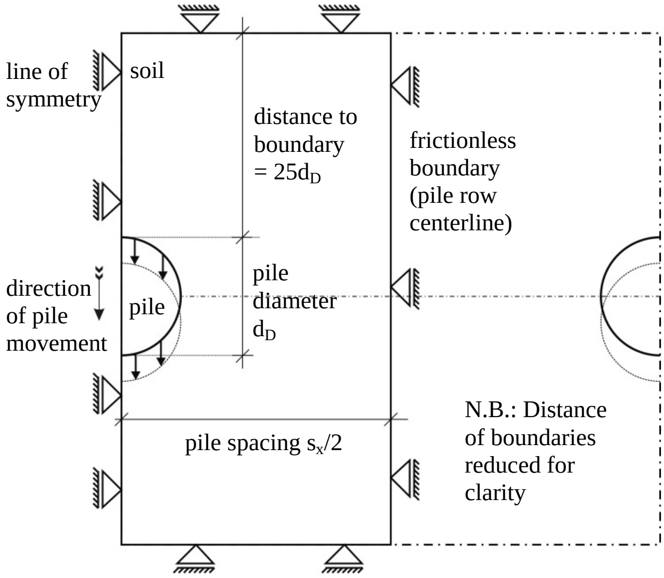


Figure 9 Boundary conditions for benchmark problem of passive loaded pile

4.3.3 Results

Typical load-displacement curves with a normalised load ($p/c_u\ d_D$) obtained from the Finite Element analyses are shown in Figure 10 for s_x/d_D = 25 (single pile) and several adhesion factors α. For the case of a single pile, bearing capacity factors N_p of 9.2, 10.5 and 11.9 were calculated for adhesion factors α of 0.0 (smooth), 0.5 (medium rough) and 1.0 (fully rough) respectively. These values compare excellently with the exact solution by Randolph & Houlsby (1984) (0.5%, 0.3 and 0.4% difference).

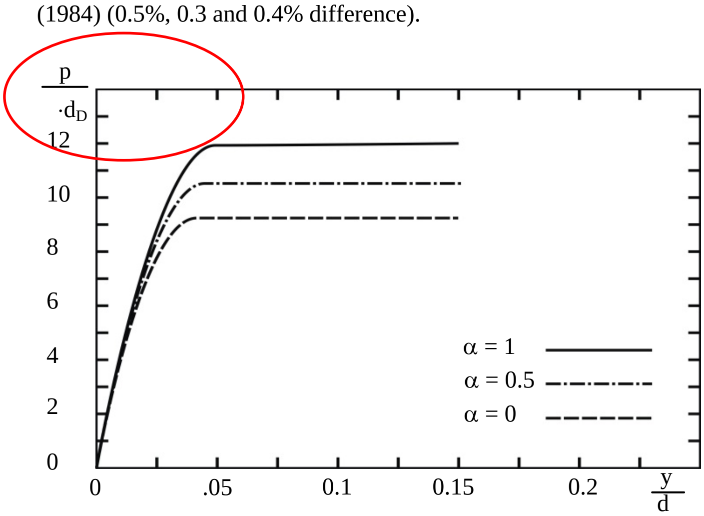


Figure 10 Load displacement curves for a single pile with various adhesion factors α

When the pile is displaced and passive load on pile increases, soil deformation mechanisms change continuously. Figure 11 shows the incremental soil displacement vectors at the plastic limit for a fully rough pile. The slip plane shows exactly the deforming zone according to Randolph & Houlsby (1984).

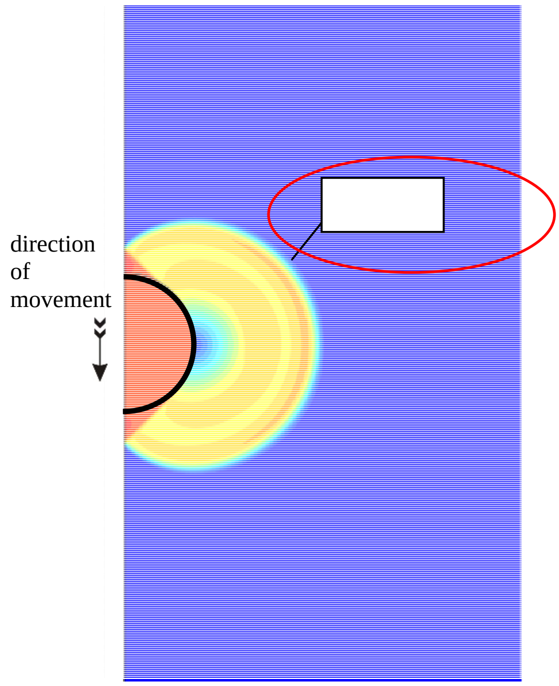


Figure 11 Contours of incremental displacements for fully rough pile with α = 1.0 (UL-FEM)

In Figure 12 the maximum and minimum principle total stresses for a single pile are shown. The numerical model is able to capture the rotation of principle stresses in front and behind of the pile nicely.

In the ALE and CEL calculations the pile was moved up to approximately 1.0 m into the soil. Instead of a prescribed displacement, a low velocity of 0.5 m/s was applied to the rigid pile so that this problem seems to fulfil the requirements of quasi-static analysis using a rate-independent constitutive model. An explicit algorithm was used to solve this problem. The soil was also allowed to separate from the pile (gap). Figure 14 shows results of the numerical calculations. Both methods led to fundamentally comparable results concerning stresses and developing of a gap behind the pile, although both methods are based on fundamental different numerical approach.

In addition, analyses have been performed with an MPM code that has been developed to solve large deformation problems. Figure 14c shows results of an MPM calculation with explicit time integration. Rough contact was considered.

In case of a quasi-static calculation, there is no difference in numerical results if the pile or the surrounding soil is moving. However, where dynamic effects play a major role, numerical results may diverge for both cases and special consideration has to be paid to absorbing boundaries or damping. In the MPM calculation a force was applied to the piles.

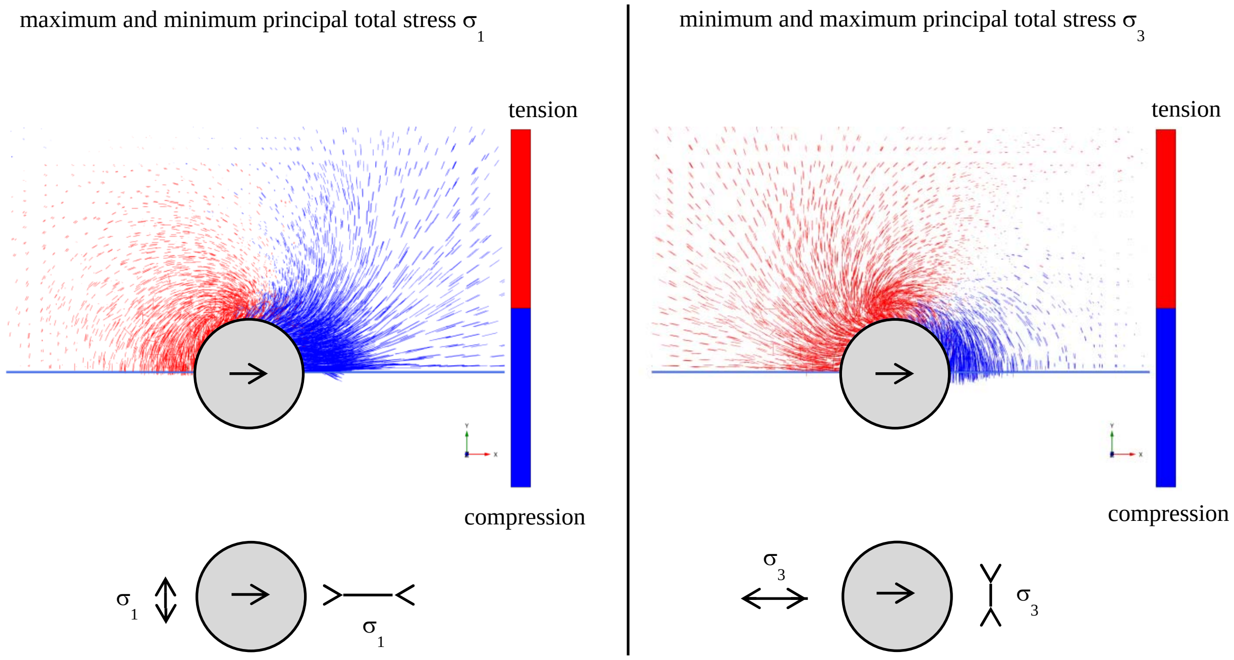


Figure 2 Maximum and minimum principle total stresses for a single pile

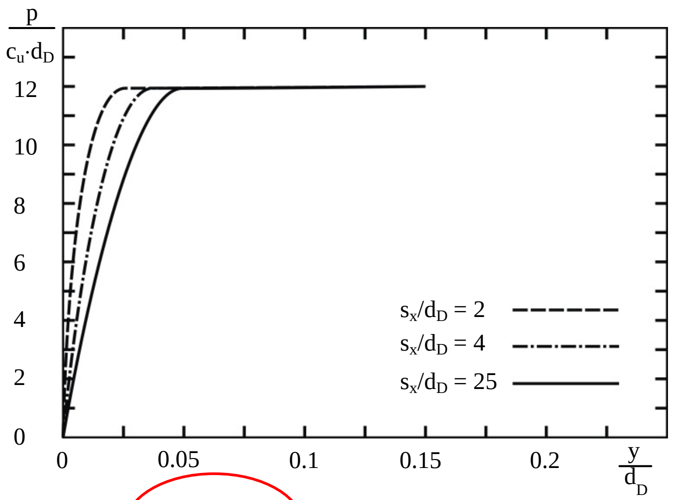
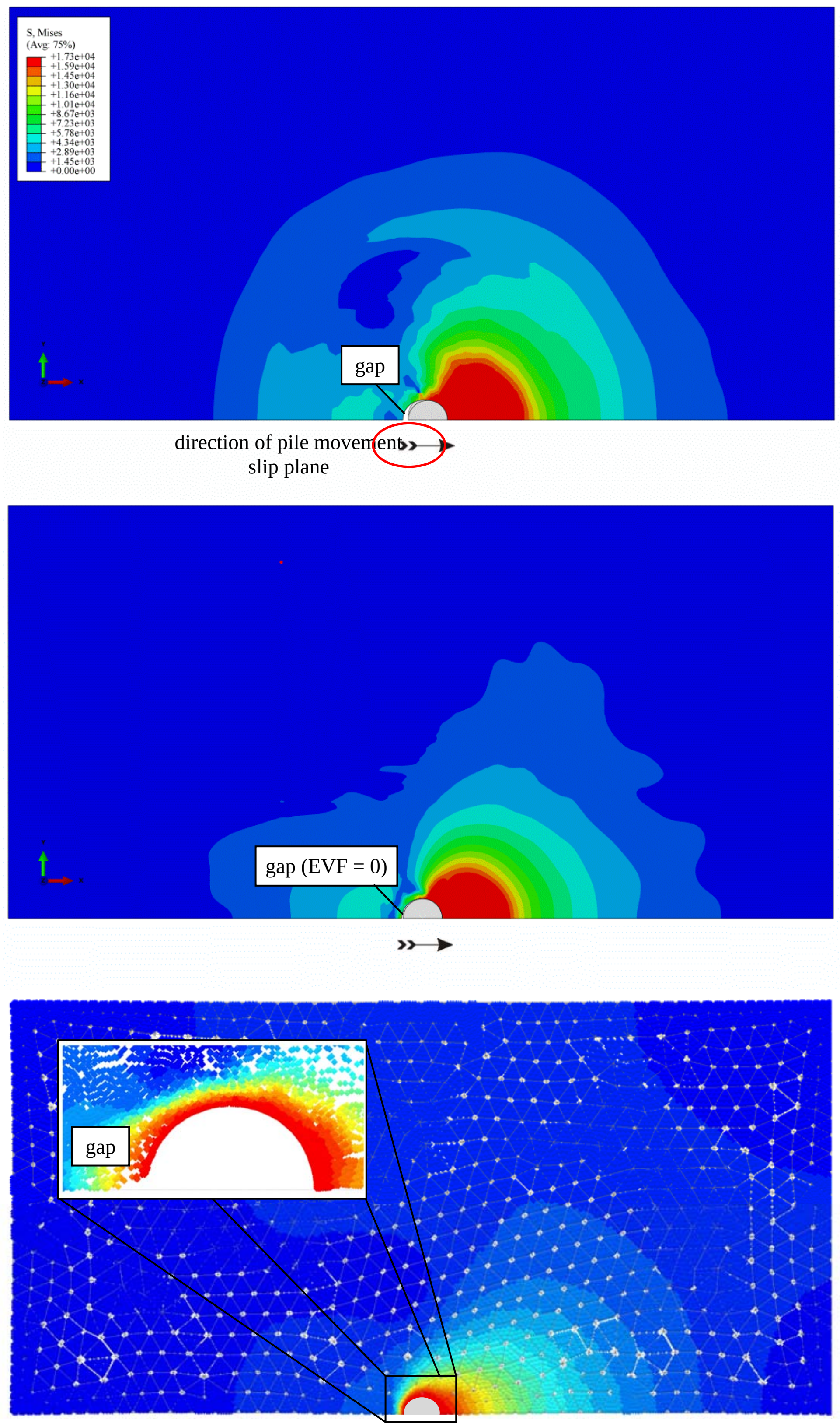


Figure 3 Different s_x/d_D pile spacings

a)

b)

c)



)

Figure 14 Numerical results of a pile moving in horizontal plane; 2D-slice of soil with gap: a) ALE, b) CEL (shown: stresses) and c) MPM calculations (shown: total displacements u)

5. CONCLUSION AND OUTLOOK

An important application in the geotechnical field has been tackled, i.e. the flow of soil around a pile. The benchmark test comprises 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.

The widely-used Updated Lagrangian Finite Element Method (UL-FEM) shows its limitations when modelling flow around a pile. For the adequate numerical modelling of large displacements (e.g. dowels in moving ground) expertise and advanced modelling techniques are required. The Arbitrary Lagrangian-Eulerian (ALE) method, the Coupled Eulerian-Lagrangian (CEL) method and the Material Point Method can overcome mesh distortion and contact problems. All methods have the potential to improve the understanding of the complex interactions but also to optimize piled foundations by numerical tools. Especially the following aspects have to be considered whenever numerical analyses with the Arbitrary Lagrangian-Eulerian (ALE) method and the Coupled Eulerian-Lagrangian (CEL) method are used:

- ALE and CEL methods are successfully applied to model large deformations (e.g. flow of soil around a pile).
- When dealing with geomechanical problems, the ALE method is probably more flexible and advantageous than the CEL method since:
  - -only the 3D 8-node element is available for Eulerian Elements
  - -mass scaling is not supported for Eulerian elements
  - -accuracy of an Eulerian model is slightly less than that of a Lagrangian model with the same mesh density
  - -advanced contact formulations are available for ALE method.
- Both methods are able to model a gap behind the pile nicely.

Like the ALE and CEL method, the Material Point Method (MPM) is able to capture the physical problem of the lateral flow mechanism of soil around a pile. Due to the undrained soil conditions, the following aspects have to be considered in the future:

- Application of a 2-phase-formulation which allows to model consolidation effects for e.g. dowels in slopes.
- Investigation of pile soil interaction using adhesive contact properties.

It is also important to mention that the characteristic behaviour of soft soil under large deformations should take into account effects like strain softening, strain rate effects or density change. However, for geotechnical applications with large deformations more advanced soil models are needed.

6. REFERENCES

Aschrafi. J., Mühl, A., Glockner, A., Moormann, C. (2013) "Numerische Untersuchungen zur passiven Horizontalbeanspruchung von Pfählen in weichen bindigen Böden", Mitteilungen des Instituts für Grundbau und Bodenmechanik, Technischen Universität Braunschweig, Heft 96, S. 283-304.

Aussilio, E., Conte, E., Dente, G. (2001) "Stability analysis of slopes reinforced with piles", Computers and Geotechnics, Vol. 28(2001), pp591-611.

Bathe, K.-J. (1996) "Finite Element Procedures". Prentice Hall.

Benson, D. J. (1989) "An efficient, accurate, simple ALE method for nonlinear FE programs", Computer Methods in Applied Mechanics and Engineering, 72(3); pp3005-350.

Benson, D. J. (1992) "Computational methods in Lagrangian and Eulerian hydrocodes", Computer Methods in Applied Mechanics and Engineering, 99, pp235-394.

Benson, D. J., Okazawa, S. (2004) "Contact in a multi-material eulerian finite element formulation", Computer Methods in Applied Mechanics and Engineering, 193, pp4277-4298.

Beuth, L. (2012) "Formulation and application of a Quasi-static Material Point Method", Institute for Geotechnical Engineering, University of Stuttgart, PhD thesis.

Brinch Hansen, J., Lundgren, H. (1960): Hauptprobleme der Bodenmechanik. Berlin, Göttingen, Heidelberg: Springer 1960.

Bransby, M. F. (1996) "Difference between load-transfer relationship for laterally loaded pile groups: active p-y or passive p-δ", Journal of Geotechnical Engineering, ASCE, Vol. 122(12), pp1015-1018.

Bransby, M. F., Springman, S. (1996) "3-D Finite element modelling of pile groups adjacent to surcharge loads", Computers and Geotechnics, Vol. 19(4), pp301-324.

Bransby, M. F., Springman, S. (1997) "Centrifuge modelling of pile groups adjacent to surcharge loads", Soils and Foundations, Vol. 37(2), pp39-49.

Bransby, M. F., Springman, S. (1999) "Selection of load-transfer function for passive lateral loading of pile groups", Computers and Geotechnics, Vol. 24(1999), pp155-184.

Broms, B. B. (1964) "Lateral resistance of piles in cohesive soils", J. Soil Mech. Fdns Div. Am. Soc. Civ. Engrs 90, SM2, pp27-63.

Cai, F., Ugai, K. (2000) "Numerical analysis of the stability of a slope reinforced with piles", Soils and Foundations, Vol. 40(1), pp73-84.

Cai, F., Ugai, K. (2011) "A subgrade reaction solution for piles to stabilise landslides", Geotechnique 61, No. 2, pp143-151.

Chaoui, F., Magnan, J. P., Mestat, P. (1994) "Three-dimensional analysis of the behaviour of piles in unstable slopes", Computer Methods and Advances in Geomechanics, pp2297-2302.

Chen, C.-Y., Martin, G. R. (2002) "Soil-structure interaction for landslide stabilizing piles", Computers and Geotechnics, Vol. 29(2002), pp363-386.

Dassault Systéms, Abaqus/Explicit (2013), Users´s manual (Version 6.13).

Ellis, E. A., Springman, S. M. (2001) "Full-height piled bridge abutments constructed on soft clay", Geotechnique, Vol. 51(1), pp3-14.

Ellis, E. A., Durrani, I. K., Reddish, D. J. (2010) "Numerical modelling of discrete pile rows for slope stability and generic guidance for design", Geotechnique, Vol. 60(3), pp185-195.

Eurocode 7-1 (EN 1997-1:2009).

Fırat, S. Sarıbıyık, M., Çelebi, E. (2005) "Lateral load estimation from visco-plastic mud-flow around cylindrical row of piles", Applied Mathematics and Computation, Vol. 173(2006), pp803-821.

Gudehus, G., Schwarz, W. (1984) "Stabilisierung von Kriechhängen durch Pfahldübel", Baugrundtagung 1984 in Düsseldorf, Essen: Deutsche Gesellschaft für Erd- und Grundbau e.V., S. 669-681.

Georgiadis, K.; Sloan, W.; Lyamin, A.V. (2013) "Ultimate lateral pressure of two side-by-side piles in clay". Geotechnique, Vol. 63(9), pp733-745.

Goldscheider, M., Gudehus, G. (1974) "Verbesserte Standsicherheitsnachweise", Baugrundtagung 1974 in Frankfurt/Main-Höchst. Essen: Deutsche Gesellschaft für Erd- und Grundbau e.V., S. 99-118.

Henke, S., Qui,G., Grabe, J.(2010) "A coupled Eulerian-Lagrangian approach to solve geotechnical problems involving large deformations", Proc. Of 7th European Conference on Numerical Methods in Geotechnical Engineering (NUMGE) in Trondheim/Norway, pp. 233-238.

Hirt, C. W., Amseden, A. A., Cook, J. L. (1974) "An arbitrary Lagrangian-Eulerian computing method for all flow speeds", Journal of Computational Physics, Vol. 14(3), pp227-253.

Ito, T., Matsui, T. (1975) "Methods to estimate lateral force acting on stabilizing piles", Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 15(4), pp. 43-59.

Ito, T., Matsui, T., Hong, W. P. (1982) "Extended design method for multi-row stabilizing piles against landslide", Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 22(11), pp. 1-13.

Jeong, S., Seo, D., Lee, J., Park, J. (2004) "Time-dependent behaviour of pile groups by staged construction of an adjacent embankment on soft clay", Ca. Geotechn. J., Vol. 41(2004), pp644-656.

Kafaji, I. (2013) "Formulation of a dynamic material point method (MPM) for geomechanical problems", Institute for Geotechnical Engineering, University of Stuttgart, PhD thesis.

Kelesoglu, M. K., Springman, S. M. (2011) "Piled bridge abutments on soft soils: Physical and 3D numerical modelling", Proceedings of the 15th European Conference on Soil Mechanics and Geotechnical Engineering, pp885-890.

Kanagasabai, S., Smethurst, A., Powerie, W. (2011) "Three-dismensional numerical modelling of discrete piles used to stabilize landslides", Can. Geotechn. J., Vol. 48(2011), pp1393-1411.

Khoei, A., Anahid, M., Shahim, K., DorMohammadi, H. (2008) "Arbitrary lagrangian-eulerian method in plasticity of pressure-sensitive material: application to powder forming processes", Computational mechanics, Vol. 42, pp13-38.

Leussink, H. Wenz, K. P. (1969) "Storage yard foundations on soft cohesive soils", Proceedings of the 7th International Conference of Soil Mechanics and Foundation Engineering, Mexico City, Mexico, 25-29 August 1968, A.A. Balkema, Rotterdam, The Netherlands, Vol. 2(1969), pp149-155.

Miao, L. F., Goh, A. T. C., Wong, K. S., The, C. I. (2006) "Three-dimensional finite element analysis of passive pile behaviour", International Journal of Numerical and Analytical Methods in Geomechanics. Vol. 30(2006), pp599-613.

Moormann, C., Katzenbach, R. (2002) "Numerical analysis of deep excavations reinforced by foundation piles", Proc. 8th Int. Symposium on Numerical Models in Geomechanics, NUMOG VIII, 10-12 April 2002, Rome, Italy, Pande, G.N. & Pietruszczak, S. (eds.), Balkema, Swets & Zeitlinger, Lisse, pp529-534.

Mühl, A., De Carvalho, P., Glockner, A., Röder, K. (2011) "Instrumentation and monitoring of a full-scale test on pile foundations for overhead bridge cranes", 8th International Symposium on Field Measurements in GeoMechanics, Berlin, 12.-16.09.2011.

Noh (1964) "CEL: A time-dependent, two-space-dimensional, coupled eulerian-lagrangian code", Methods in Computational Physics, Vol. 3, pp117-179.

Pan, J. L., Goh, A. T. C., Wong, K. S., Selby, A. R. (2002) "Three-dimensional analysis of single pile response to lateral soil movements", International Journal of numerical and analytical Methods in Geomechanics. Vol. 26(2002), pp747-758.

Poulos, H. G. (1995) "Design of reinforcing piles to increase slope stability", Can. Geotechn. J., Vol 32(1996), pp808-818.

Oteo, x. (1977) "Horizontally loaded piles – deformation influence", Proc. 9th ICSMFE, Spec. 10, Tokyo, pp101-106.

Qui, G., Henke, S., Grabe, J. (2010) "Application of a Coupled Eulerian-Lagrangian approach on geomechanical problems involving large deformations", Computers and Geotechnics, Vol. 38(1), pp30-39.

Qui, G. (2012) "Coupled Eulerian Lagrangian simulation of selected soil-structure interaction", Institut für Geotechnik und Baubetrieb, Technische Universität Hamburg-Harburg", PhD thesis.

Randolph M.F. and Houlsby, G.T. (1984) "The limiting pressure on a circular pile loaded laterally in cohesive soil", Geotechnique, London, Vol. 34(4), pp613-623.

Recommendations on Piling (EA-Pfähle) (2013), Ernst & Sohn Series on Geotechnical Engineering, 2nd edition.

Schwarz, W. (1987) "Verdübelung toniger Böden", Institut für Boden- und Felsmechanik der Universität Karlsruhe, Heft 105.

Smoltczyk, H.-U. (1966) "Pfahlroste, Berechnungen und Ausbildung", In Grundbau-Taschenbuch, Bd. 1, 2. Auflage, Ernst u. Sohn, Berlin, S. 658-715.

Stewart, D. P., Jewell, R. J., Randolph, M. F. (1994) "Design of piled bridge abutments on soft clay for loading from lateral soil movements", Geotechnique, Vol. 44(2)2, pp277-296.

Tian, Y., Wang, D., Cassidy, M. J. (2011) "Large deformation finite element analysis of offshore penetration tests", Proc. 2nd Int. Symp. on Computational Geomechanics, Cavtat-Dubrovnik, pp.925-933.

Wenz, K. P. (1963) "Über die Größe des Seitendrucks auf Pfähle in bindigen Erdstoffen", Institut für Boden- und Felsmechanik der Universität Karlsruhe, Heft 12.

Winter, H. (1979) "Fließen von Tonböden. Eine mathematische Theorie und ihre Anwendung auf den Fließwiderstand von Pfählen", Institut für Boden- und Felsmechanik der Universität Karlsruhe, Heft 82.

Xu, K. J., Poulos, H. G. (2001) "3-D elastic analysis of vertical piles subjected to "passive" loadings", Computers and Geotechnics, Vol. 28(2001), pp349-375.

Zienkiewicz, O. C., Taylor, R. L. (1967) "The Finite Element Method", New York: McGraw-Hill.