GROUP EFFECTS FOR PILE ROWS UNDER PASSIVE LATERAL LOADING

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ABSTRACT

Ground movements in soft cohesive soils impose lateral pressure on piles. Among others, here the magnitude of pressure depends on the amount and speed of ground movement, the pile's bending stiffness and the geometrical boundary conditions. In recent decades, considerable effort has been devoted by many researchers to study the problem of lateral pressure on piles in soft cohesive soils, experimentally and using numerical methods. Unfortunately, these investigations have led, for example, to highly divergent calculation approaches, e.g. pile group factors.

In classical *Lagrangian* Finite Element Method, large soil displacements distort the finite element mesh which might lead to inaccurate results or the numerical analysis does not converge to a stable solution. Therefore, most numerical based research on lateral loads was done for moderate soil displacements. Numerical investigations of the problem are more challenging when large deformation respectively is involved. In continuum-based models, the traditional description of kinematics is either based on *Lagrangian* or *Eulerian* approach where each has its own advantages and disadvantages. Coupling the two descriptions in one approach by exploiting the best features of each is therefore desirable. One coupling approach, called the Material Point Method (*MPM*) will be explained in brief. The potential of the method is shown with respect to the results of a small-scale test on a single pile in soft clay. Furthermore, numerically calculated lateral pressure and group effects will be discussed.

Keywords: piles, lateral thrust, passive, soil-pile-interaction, CPDI-method

INTRODRUCTION

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 or groups are loaded in the lateral direction, e.g. by creeping soil has 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, Ellia & Springman 2001).

Determining the magnitude of the passive lateral loads requires not only knowledge of the material properties, but also of the stress fields 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 non-linear 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, the limiting pressure on a single pile was determined to be between 7*cu* and 10*cu*, where, *cu* is the undrained shear strength of the soil. Broms (1964) proposed a limiting pressure of 9*cu*. This 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 2*cu* and 5*cu*. In the analysis by Randolph & Houlsby (1984), it was found that the values of ultimate soil pressure ranges roughly between 9*cu* to 12*cu*. However, in the back-analysis of piles in unstable slopes conducted by Vigiani (1981), the ultimate soil resistance was found to be somewhat lower, i.e., between 2.8*cu* and 4*cu*. Poulos (1995) indicated that the limiting lateral soil pressure increases linearly from 2*cu* at the ground level to 9*cu* at a depth 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 11.75*cu*, and thus slightly smaller than the value of 12.5*cu* 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.

Figure 1 shows the displacement field of the soil around a pile, the compressive forces of the soil along with its components, the stress field on the pile on a plane, and along the length of the pile.

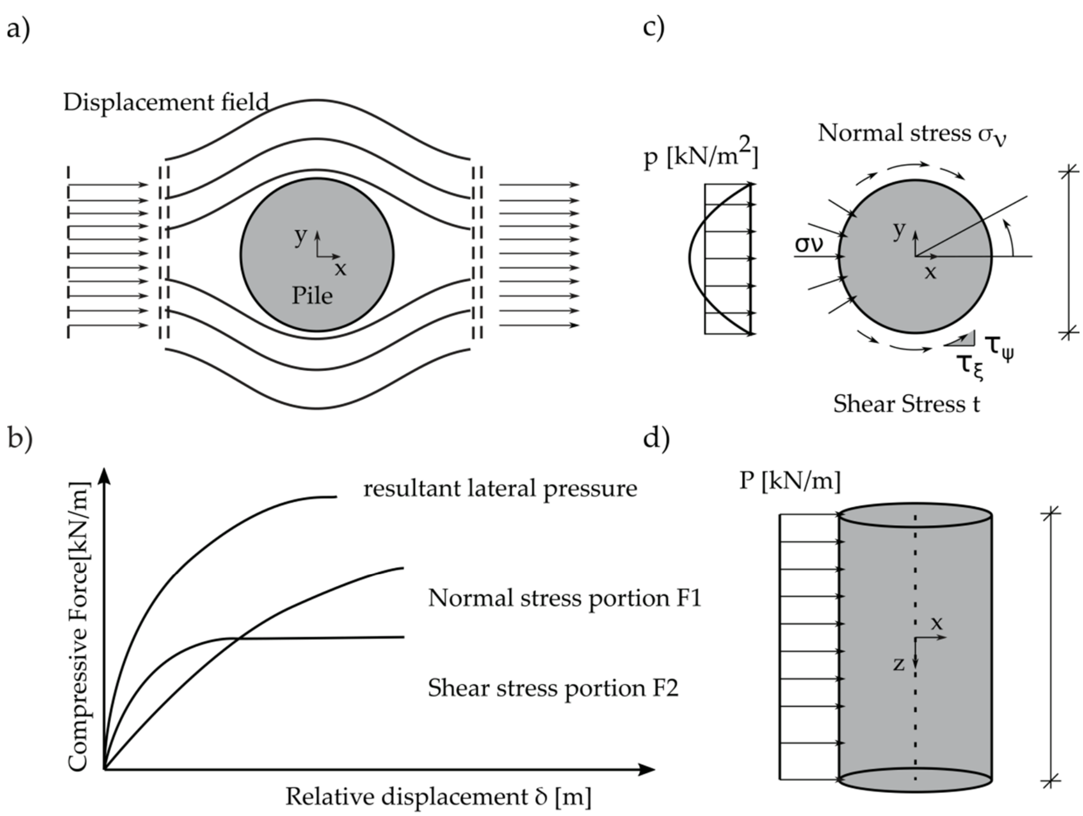

Fig. 1. General definitions for passive piles

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 to the problem. For this reason, use of more sophisticated modelling techniques is being sought out.

NUMERICAL APPROACHES

The Material Point Method *(MPM)* is a particle-based method that combines both *Eulerian* and *Lagrangian* procedures to model large deformation problems. In *MPM*, the continuum is represented by *Lagrangian* points, also called as material points, which move through a fixed *Eulerian* mesh. The physical properties of the continuum such as mass, momentum, strains, stresses as well as state variables are stored in the particles, whereas the *Eulerian* mesh carries no permanent information. At the beginning of the computational time step, the information is transferred from the particles to the computational mesh. Incremental solution of governing equation is then determined in a *Lagrangian* fashion on the mesh. At the end of the computation step, the solution is mapped back to the particles from the mesh, updating the information. Figure 1. represents the sequence of events for one computational step in solution using material point method.

This approach in *MPM* combines the best aspects of both *Lagrangian* and *Eulerian* formulations, at the same time, overcoming some shortcomings in them. The problem of mesh distortion which is present in a calculation using the updated *Lagrangian* solution for large deformation is avoided. In spite of its advantages over other methods, *MPM* has shown to exhibit numerical difficulties during extreme deformation, causing oscillation in stress state. To overcome this difficulty, the classic *MPM* formulation has been improved over the years, specifically by method in which the concentrated mass of the material point is distributed over a finite subdomain.

CONVECTED PARTICLE DOMAIN INTERPOLATION METHOD (CPDI)

During large material movements of the continuum, the particles cross the computational grid, causing severe oscillations in the stresses. This occurs when linear shape functions are used, causing the gradients to be discontinuous at the interfaces, leading to unbalanced forces on the common shared nodes. This is referred to as the grid-crossing or cell-crossing error. By increasing the number of particles per cell, or by using a higher order shape function, the oscillations can be reduced. This consequently increases the computational load for the solution, and is therefore, not feasible. Instead, by using the same number of particles and linear shape functions, Bardenhagen & Kober (2004) proposed the Generalized Interpolation Material Point (*GIMP*) method, wherein the concentrated mass of the particle is spread over a finite square domain. A later improvement in *GIMP* was brought about in the form if *cpGIMP* in which the axial update of the geometry configuration was allowed. Sadeghirad et al. (2011) proposed a procedure where the domain is updated according to the particle deformation. The initial domain is parallelogram shaped, and its sides are continuously updated using the deformation gradient. This version of *MPM* was named as Convected Particle Domain Interpolation (*CPDI*). Figure 2 highlights the differences in the domains between the various versions of *MPM*. In this work, an axisymmetric CPDI procedure has been used. A more rigorous explanation for the formulation of *CPDI* is presented in the work of Hamad (2016).

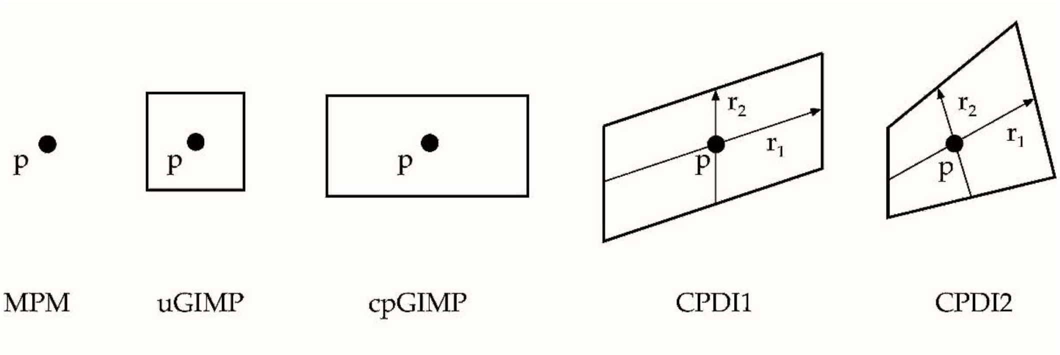

Fig. 2. Versions of Material Point Method

Standard *MPM* formulations provides an automatic no-slip condition. In the classic sense of contact in *MPM*, contact is detected when the material points of different entities contribute to the same grid node of the computational mesh. Therefore, interaction is activated before the actual contact taking place. Furthermore, a lack of smoothening function also contributes to oscillations in contact stresses. In the present implementation, a penalty function method that is often implemented in the FE analysis is introduced in *CPDI*, where the contact force is assumed proportional to the residual of the impenetrability constraints and the surface stiffness.

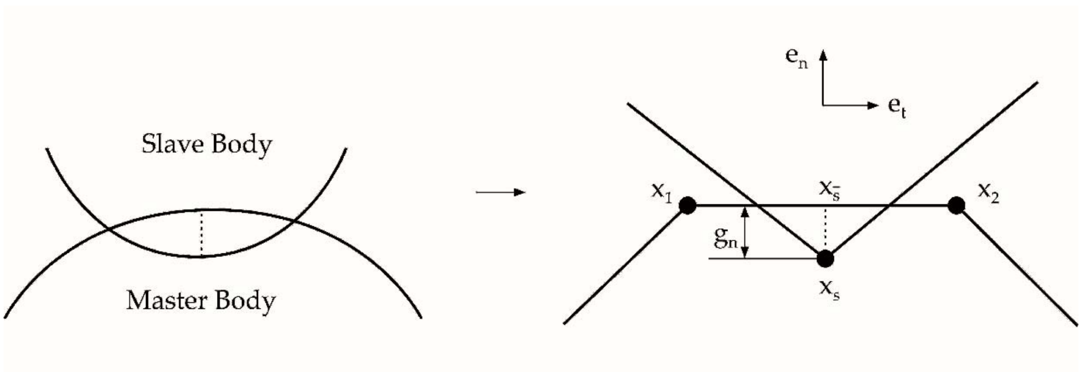

Fig. 3. Penalty contact conditions

The surface of the continuum in *MPM* is discretized separately from the volume discretisation. By setting an amount of mass to the interface, the surface nodes are able to follow the deformation of the continuum. Upon the equation of motion, the surface nodes of individual entities might interact according to the penalty function. Frictional forces are then traced back as an external contact force acting on the boundary. Early works focussed on relaxing the no-slip and no-separation condition Bardenhagen, et al. (2001). Subsequent works have been carried out to incorporate coulomb friction in the contact formulation and has been applied to geotechnical applications like pile installation successfully Hamad (2016). As an improvement to the contact algorithm, the penalty function method is introduced by Hamad et al. (2017) to evaluate the contact forces between interacting bodies. In the penalty function method, if a region Γ_c where contact violation exists, as shown in Figure 3, the potential energy is penalised proportional to the amount of constraint violation by using a penalty function P, expressed as

$$P = \frac{1}{2}\omega_n \int_{\Gamma_c} {g^2}_n d\Gamma_c + \frac{1}{2}\omega_t \int_{\Gamma_c} {g^2}_t d\Gamma_c, \quad [1]$$

where, ω is the penalty parameter, g is the gap function, and the subscripts n and t are the normal and tangential directions, respectively.

In this method, the interface of the continuum is discretised using two-node linear segments. Normal and tangential stiffness is assigned to these elements, which are here, the penalty parameters in the formulation. A distinction between master and slave is made in the formulation for numerical convenience. Detailed formulation of the algorithm is presented in the work of Hamad et al. (2017).

BENCHMARK PROBLEM

To evaluate the methods presented, the numerical prediction of the bearing capacity of a strip footing is chosen, owing to the availability of an analytical solution. In addition to evaluating the *CPDI* method, various other methods used in *ABAQUS* is also evaluated. The analytical solution of the problem is given by

$$q = (2 + \pi)c, \qquad [2]$$

where, q represents the footing pressure at collapse, and c, the cohesion and is chosen as 100 kPa for all the calculations.

Figure 4 shows the results of *CPDI* compared with the analytical solution. Other results have been obtained from Qui (2012). It is evident that the results from *CPDI* and *CEL* simulation are closer to the analytical solution. This, therefore leads us to choose these two methods for further simulation in this work.

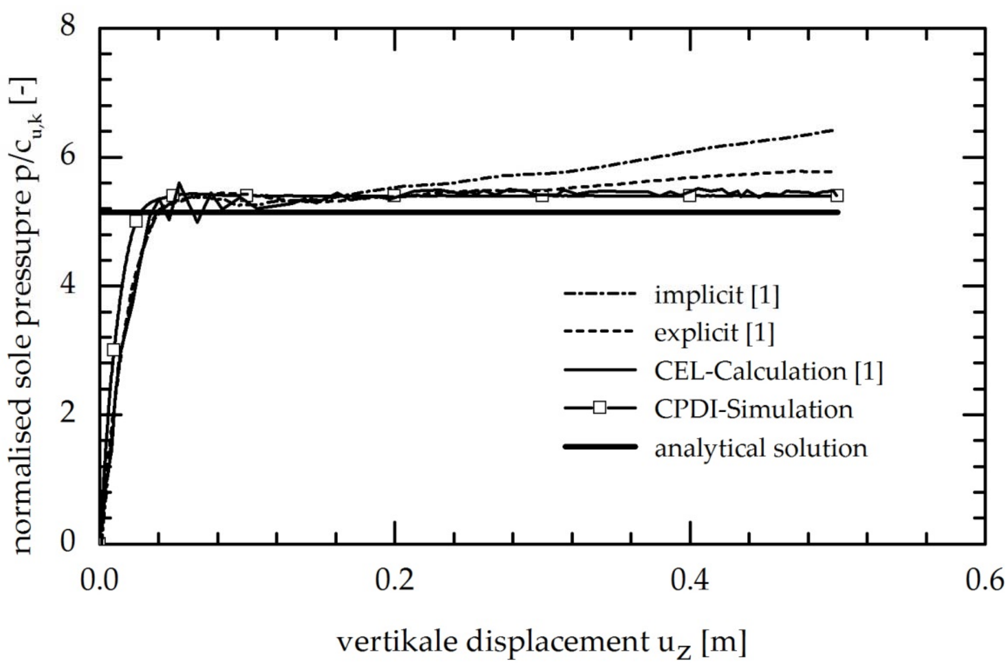

Fig. 4. Load-displacement-curves

NUMERICAL SIMULATION

The numerical simulation of the displacement of pile in soil deals with large deformations. A standard *Lagrangian* Finite Element simulation would fail, because of the high mesh distortion. To overcome this problem, a new numerical method, the Convected Particle Domain Interpolation (*CPDI*) method is used. Hamad (2016) and Hamad et al. (2016) have shown the potential of *CPDI* method for geotechnical applications undergoing large deformations. To simulate the behaviour of the soil, a Mohr-Coulomb constitutive law is utilised.

Constitutive model

Although soil behaves rather non-linear when subjected to changes in stress and strains, and this behaviour can be replicated using advanced soil models. For this work, however, the well-known Mohr-Coulomb model, which is a linear elastic perfectly plastic model is chosen. The linear elastic part of the model is based on the Hooke's law of isotropic elasticity, and the perfectly plastic part is based on the Mohr-Coulomb criterion, formulated in a non-associated plasticity framework.

The constitutive parameters for Kaolin-clay used in this paper are listed in Table 1.

Table 1. Material parameters for Kaolin-clay (Bauer et al. 2016)

		small scale tests		
parameter	**unit**	V1	V2	V3
γ / γ	[kN/m^3]	18/18	18/18	18/18
E_u	[kN/m^2]	125	125	125
ν_u	[-]	0,495	0,495	0,495
φ	[°]	0	0	0
c_u	[kN/m^2]	2,3	2,4	2,5
ψ	[°]	0	0	0
K_0	[-]	0,685	0,685	0,685

The settlement of the pile is needed to activate the frictional and bearing resistance, and it is assumed that the resistance of the pile is fully activated after a settlement of 0.1D.

Numerical model

The geometry of the numerical model is based off of the geometry of the scaled model test. The dimensions of the model, along with the orientation of the pile groups are shown in Figure 5.

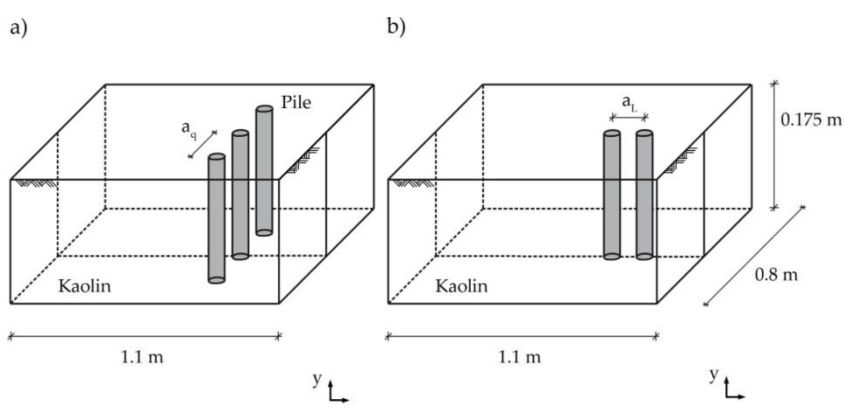

Fig. 5. Geometrical configuration: a) pile group in horizontal rows, b) pile group in transverse row

Results

Figure 6 shows the contour plot of the total horizontal deformation u_x [m] for a pile row consisting of 2 piles with varying center distances, 2D and 10D, respectively, at different soil displacement distances at $0.5d_D$ and $1d_D$, respectively.

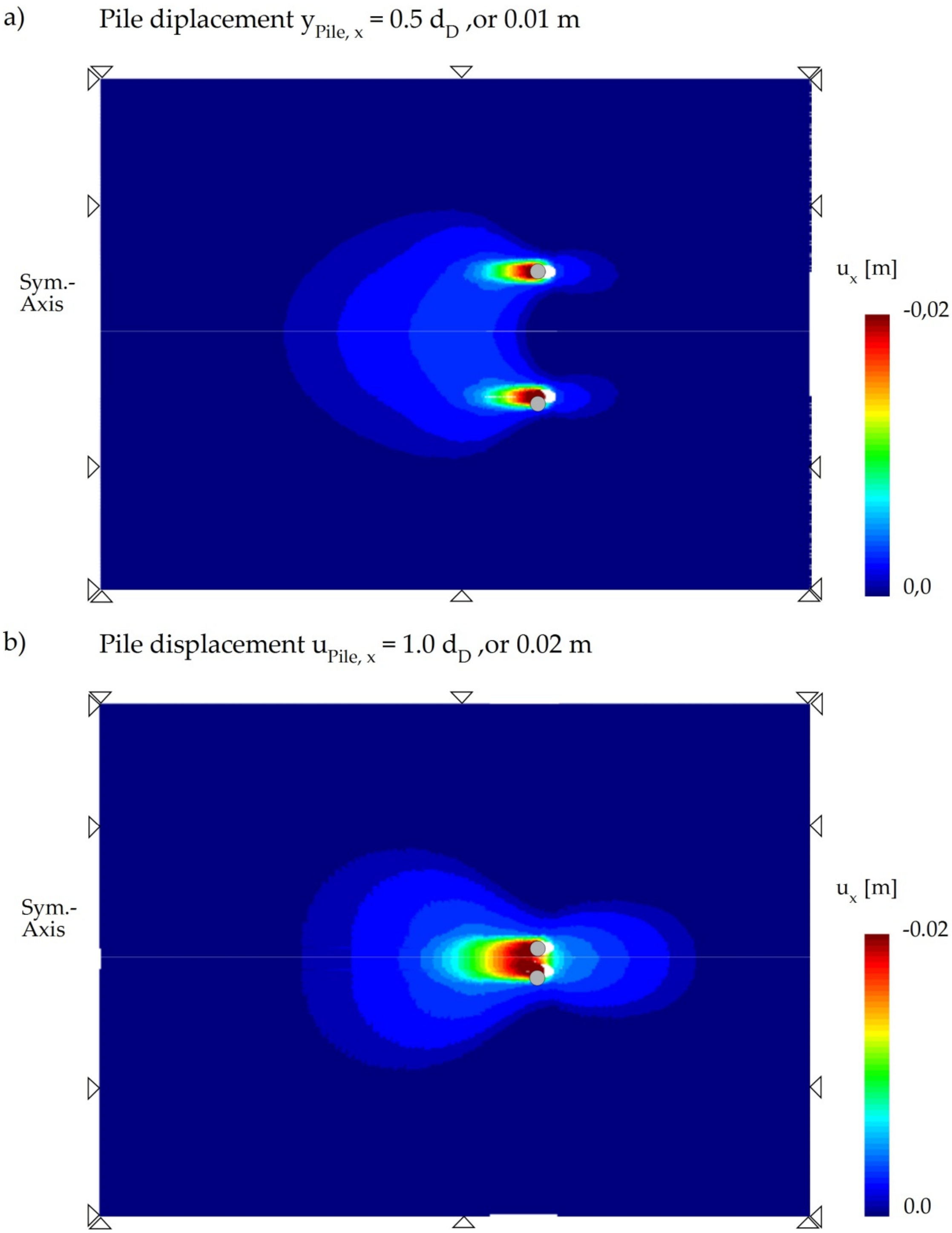

Fig. 6. Displacement of soil around pile a) center distance 10D, b) center distance 2D

Figure 6 (a) shows the horizontal displacement field for a normalised centre distance of 10D, with a normalised horizontal displacement of $0.5d_D$, i.e., 0.01 [m]. In Figure 6 (b) shows the results of displacement field for an axis distance of 2D, for a horizontal pile displacement of $1d_D$, i.e., 0.02 [m]. With the pile group for a smaller value for the axis distances, the mutual dependence of the piles on each other is clearly observable. The total displacement fields of the soil overlap in the vicinity of the individual piles. As the centre distance a_q of the pile row group increases, the mutual dependence of the piles on their displacement fields decreases significantly, as shown in Figure 6 (b) In case of widely spaced piles, i.e., for an axis distance more than 10D, the corresponding behaviour would resemble the behaviour of a single pile.

Figure 7 shows the results of the normalised force-displacement curve from the numerical simulation of a transverse pile rows, and is compared with the results from the experimental simulation. The calculations are carried out for two centre distances, 2D and 10D, respectively.

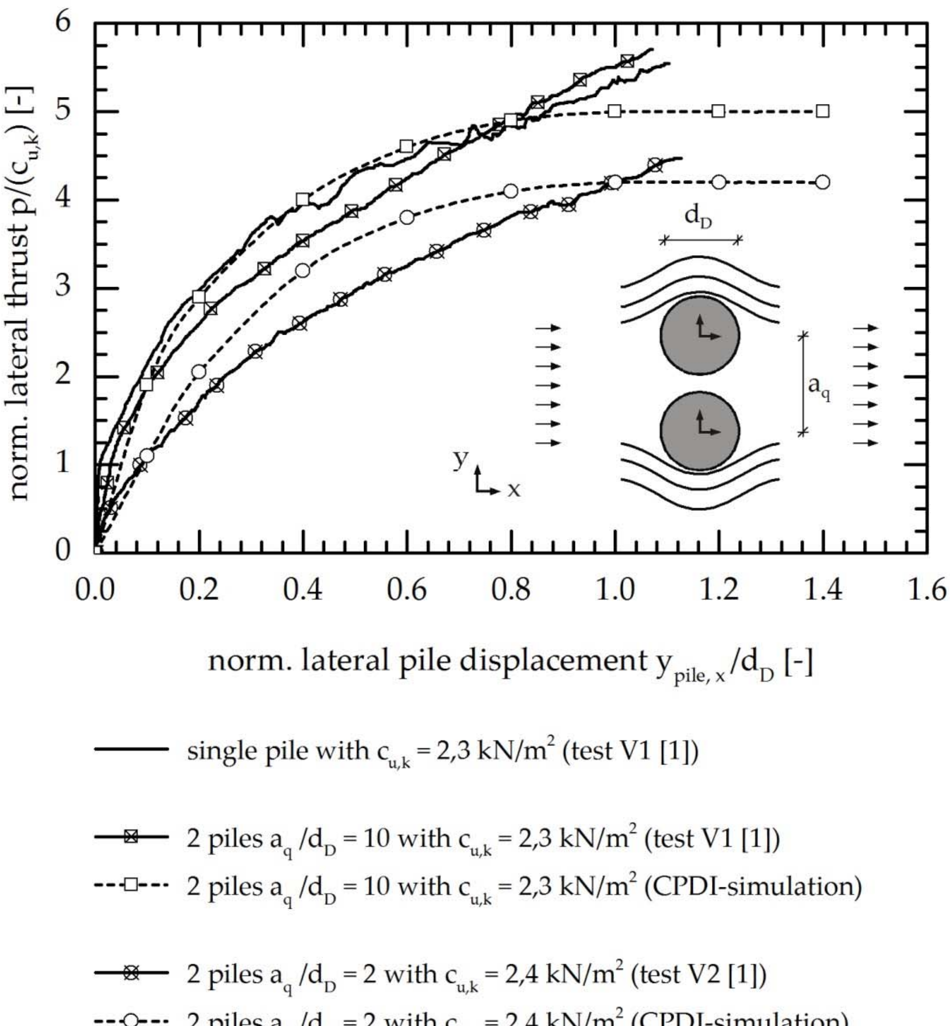

Fig. 7. Results for two piles in transverse configuration, comparison of test results and simulation

The results for the two-pile row, with varying centre distances, from numerical simulation is presented, and is compared with the experimental results for pile group, along with the result for a single pile. In this work, the normalised pressure is calculated as an average of the force measured on both the piles. It is observed that with increasingly narrower centre distances, the normalised force-displacement curve starts to resemble that of a single pile. In addition, it is observed that there is an increasingly less rigid behaviour of the experimentally measured and numerically determined load-deflection curves. For an axis distance of $a_d = 10D$, the normalised load-displacment curve of the pile group and the single pile are nearly superimposed. In this axis distance, the pile group behaves almost like a single pile. As with the single pile, a limit state is formed, due to the chosen constitutive model for the soil, for which in experiments, the soil displacement of 0.02 [m] or $1d_D$ could not be determined. For a pile displacement of more than $1d_D$, the results from experiments and the results from the numerical simulation using *CPDI* start to deviate. To conclude, it can be said that the case of transverse pile row, each consisting of two piles, there is a good agreement between the experimental results shown by Bauer et al. (2016) and the results from the numerical analysis.

CONCLUSIONS

In this paper, a novel method to investigate the displacement of the soil around a pile group is presented. To overcome the effects of mesh distortion due to large deformation of soil, a novel method, known as *CPDI* is adopted for numerical analysis. One cases of pile row with varying centre distances were chosen, and the results of the *CPDI* simulation were compared with the results of the experimental work. It is observed that the experimental and numerical results are in good agreement with each other, and is able to reproduce the behaviour of the soil around the pile accurately.

REFERENCES

Aussilio, E. & Conte, E., Dente, G. (2001). “Stability analysis of slopes reinforced with piles”, *Computers and Geotechnics*, 28(2001), 591-611.

Bardenhagen, S. G., & Kober, E. M. (2004). The generalized interpolation material point method. CMES: *Computer Modeling in Engineering & Sciences*, 5, 477-495.

Bardenhagen, S. G., Guilkey, J. E., Roessig, K. M., Brackbill, J. U., Witzel, W. M., & Foster, J. C. (2001). An improved contact algorithm for the material point method and application to stress propagation in granular material. CMES: *Computer Modeling in Engineering & Sciences*, 2, 509-522.

Bauer, J., Kempfert, H. G., & Reul, O. (2016). Lateral pressure on piles due to horizontal soil movement. *International Journal of Physical Modelling in Geotechnics*, 16(4), 173-184.

Bransby, M. F. & Springman, S. (1997). “Centrifuge modelling of pile groups adjacent to surcharge loads”, *Soils and Foundations*, 37(2), 39-49.

Bransby, M. F. & Springman, S. (1999). “Selection of load-transfer function for passive lateral loading of pile groups”, *Computers and Geotechnics*, 24(1999), 155-184.

Broms, B. B. (1964). “Lateral resistance of piles in cohesive soils”, *J. Soil Mech. Fdns Div. Am. Soc. Civ. Engrs 90*, 27-63.

Cai, F. & Ugai, K. (2011). “A subgrade reaction solution for piles to stabilise landslides”, *Geotechnique* 61(2), 143-151.

Chen, L. (1994). Effect of lateral soil movements on pile foundations (*Doctoral dissertation*).

Ellia, E. A. & Springman, S. M. (2001). “Modelling of soil-structure interaction for a piled bridge abutment in plane strain FEM analyses”, *Computers and Geotechnics,* 28(2001), 79-98.

Goldscheider, M. & Gudehus, G. (1974). Verbesserte Standsicherheitsnachweise, *Vorträge der Baugrundtagung 1974 in Frankfurt/Main-Höchs*t, 99-118.

Hamad, F. (2016). Formulation of the axisymmetric CPDI with application to pile driving in sand. *Computers and Geotechnics*, 74, 141-150.

Hamad, F., Giridharan, S., & Moormann, C. (2017). A Penalty Function Method for Modelling Frictional Contact in MPM. *Procedia Engineering*, 175, 116-123.

Hamad, F., Stolle, D., & Moormannm C. (2016). Material Point Modelling of releasing Geocontainers from a Barge. *Geotextiles and Geomembranes*, 44, 308-318.

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*, 22(11), 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.*, 41(2004), 644-656.

Poulos, H. G. (1995). “Design of reinforcing piles to increase slope stability”, *Can. Geotechn. J.,* 32(1996), 808-818.

Qiu, G. (2012). Coupled Eulerian Lagrangian Simulations of Selected Soil-Structure Problems (*Doctoral dissertation, Dissertation, Veröffentlichungen des Instituts für Geotechnik und Baubetrieb der TU Hamburg-Harburg*).

Randolph M.F. & Houlsby, G.T. (1984). “The limiting pressure on a circular pile loaded laterally in cohesive soil”, *Geotechnique*, London, 34(4), 613-623.

Sadeghirad, A., Brannon, R. M., & Burghardt, J. (2011). A convected particle domain interpolation technique to extend applicability of the material point method for problems involving massive deformations. *International Journal for Numerical Methods in Engineering*, 86, 1435-1456.

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*, 44(2)2, 277-296.

Vigiani, C. (1981). “Ultimate lateral load on piles used to stabilize landslides”, Proceedings. *The 10th Int. Conf. of Soil Mech. and Found. Eng.*, Stockholm, Vol. 3, 555-560.

Wenz, K. P. (1963). Über die Größe des Seitendrucks auf Pfähle in bindigen Erdstoffen. Universität Karlsruhe, *Heft 12*.

Winter, H. (1979). Fließen von Tonböden. Eine mathematische Theorie und ihre Anwendung auf den Fließwiderstand von Pfählen. Universität Karlsruhe, *Heft 82*.