Micro-piles under dynamic horizontal excitation: Field tests and Numerical Modeling

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ABSTRACT: Field tests were performed to study the performance of vertically loaded micro-pile foundations in sand under horizontal dynamic excitation. This represents a simplified method to simulate an earthquake under true scale conditions. Two foundations were tested, each consisting of 3 inclined micro-piles and a pile cap installed in dry sand. Geophones were installed on top of the pile caps and in the vicinity of the foundations to measure the oscillations in the surrounding subsoil. 3D finite element models were created for a back analysis of the field tests. A hypoplastic constitutive model was used for the description of the mechanical behavior of the subsoil. This model comprises density and the most recent deformation history as state variables and makes possible a realistic assessment of changes of density or pore water pressures, respectively, due to monotonic and alternating loading. The paper at hand describes the field tests and numerical analyses and summarizes relevant results for the dry conditions of the field tests. Additionally, simulation results for saturated conditions are presented, which are used to judge potential liquefaction effects.

1 INTRODUCTION

The results presented in this paper are part of the outcome of a joint research and development project, where the main goal is the development of earthquake-proof deep foundations consisting of micro-pile groups especially with respect to the mitigation of liquefaction. Furthermore the behavior of this deep foundation system with respect to ductility during earthquake like excitation should be observed.

For this purpose, full-scale field tests were conducted which are described in short in the following Section 2.

These tests have been accompanied by numerical analyses based on a hypoplastic constitutive model (e.g. von Wolffersdorff 1996, Niemunis & Herle 1997). This model has been successfully applied for earthquake application, e.g. in the framework of the special research filed SFB 461: “strong earthquakes” of the German Research Foundation (cf. e.g. Buehler 2006, Cudmani et al. 2004, Gudehus et at. 2004)

For the calibration of the material parameters of the constitutive model a laboratory program was conducted. The results together with the calibration results (back analyses of the relevant element tests) are presented in Section 3.

In Section 4 numerical results of cyclic element test simulations (simple shear, triaxial shearing) under drained and undrained conditions are presented to demonstrate the capacity of the applied constitutive equation.

In Section 5 a summary and outlook is given.

2 FIELD TESTS

2.1 *Subsoil conditions*

The subsoil at the test site (dump of the pit mine Jaenschwalde in Germany) consisted of fly ash of a nearby power plant with a hard consistency. For the installation and subsequent testing of the micro-pile foundations a trial pit 7 m x 17 m wide and 4 m deep was excavated and backfilled with dry slightly gravelly quartz sand (coefficient of uniformity $U = 2.7$, coefficient of curvature $C_c = 1.2$, fine content 2.2%).

After the installation of the test and support foundations CPT soundings were performed to assess the initial conditions of the backfill in terms of density.

2.2 *Test setup and conditions*

Two groups of inclined micro-pile foundations with a pile cap (3 piles with an inclination of 15° and concrete block 1 m x 1 m x 1 m each) were installed together with two support foundations (4 piles, pile cap 1,5 m x 1,5 m x 1,0 m).

The two test foundations were different with respect to the installation pressure of the injected material. After the tests the piles were excavated and the influence of the installation pressure on the achieved diameter of the micro-piles was examined.

During the test a computer controlled horizontal hydraulic jack was setup to realize a dynamic horizontal excitation of the test foundation with up to 10 Hz and a force amplitude of ± 200 kN. On top of the test foundation a standard apparatus for applying and measuring a vertical load was installed, consisting of a hydraulic jack, test beam, tension rods and anchor piles (Fig. 1) in accordance to e.g. ASTM D1143. The decoupling of the horizontal displacements of the test foundation and the apparatus for vertical loading was realized by means of a roller bearing.

Figure 1. Computer controlled hydraulic jack for rapid horizontal loading in two directions.(upper right) and overview of test setup for dynamic horizontal excitation of one pile group under constant vertical load.

The following quantities were measured during each test:

– Vertical and horizontal load
– Vertical and horizontal displacement of the test foundation
– Horizontal displacement of the support foundation
– Vertical displacements of the anchor piles
– Vertical and horizontal vibration velocities on the test foundation and surrounding subsoil

The latter was realized using triaxial geophones, which were installed on top of the test foundation and in the adjacent subsoil in different distances and depths.

All tests were performed under a static vertical load of 1000 kN representing a realistic foundation load. The horizontal excitation was varied, the frequencies ranged from 1 to 5 Hz with force amplitudes up to ± 200 kN. The foreseen maximum frequency of 10 Hz could not be achieved for these loads, since the corresponding displacements were too high, and thus the capacity limit of the hydraulic pump was reached.

Some of the results are given together with numerical results in Section 4.3.1.2.

3 CONSTITUTIVE MODELLING

3.1 *General description*

For the modelling of the mechanical behavior of non-cohesive granular material a hypoplastic constitutive model with the extension of so-called intergranular strains is used (von Wolffersdorff 1996, Niemunis & Herle 1997). It incorporates density in terms of the void ratio as a state variable and allows for small strain effects, i.e. changes in material stiffness due to changes in the direction of deformation and subsequent shear strain dependent shear stiffness due to further monotonic shearing.

It renders a realistic modelling of monotonic and cyclic behaviour of granular materials as is shown in short below (cf. Sec. 3.2 + 3.3).

3.2 *Calibration of material parameters*

A comprehensive description of the applied iterative calibration procedure is given by (Meier 2009). Here, the material parameters of the hypoplastic constitutive model and their physical meaning are described and the laboratory tests needed for calibration are mentioned. Comparisons of the actual tests with the corresponding numerical back analyses are presented:

– Limit void ratios e_{d0}, e_{c0}, e_{i0}: $e_{d0} \approx e_{min}$ is a lower bound void ratio of a grain skeleton at zero pressure, $e_{c0} \approx e_{max}$ is the void ratio in a critical state at zero pressure. Both, e_{min} and e_{max} are determined through standard index tests (e.g. according to ASTM D4254 and D4253). e_{i0} is an upper bound void ratio of a simple grain skeleton at vanishing pressure (without macropores). $e_{i0} \approx 1.15\, e_{c0}$ and $e_{d0} \approx 0.6\, e_{c0}$ are simple established estimates (Herle & Gudehus 1999).
– Critical friction angle φ_c: The critical friction angle φ_c determines the resistance of a granulate subjected to monotonic shearing in critical state. Appropriate for the determination of φ_c are drained or undrained triaxial tests or direct shear tests on initially very loose specimens ($e_0 \approx e_{max}$). A simple, fast and yet reliable estimate is the angle of repose test with dry material.
– Granulate hardness h_s and exponent n: These two parameters govern compressibility of the grain skeleton. They also influence the curve progressions of triaxial compression tests. The initial values are found by means of curve fitting of oedometric compression tests with initially very loose specimen. The two parameters are then adjusted iteratively in the course of the calibration process.

– Exponents α and β: The exponent α controls the dilatancy behavior of the material and therewith the peak friction angle is calibrated using results of drained triaxial tests with initially dense specimen. The parameter β is used to adjust the stiffness of a grain skeleton with $e < e_c$. It is calibrated with the aid of results of oedometric compression tests with initially dense specimen.

The intergranular strain parameters were first estimated based on experience and then adjusted such that the results of the cyclic oedometric loading could be reproduced (Meier 2009).

The calibration yielded the following parameters:

Table 1: Material parameters gained through calibration

Material parameters	Intergranular strain parameters
$e_{d0} = 0.652$	$R = 10^{-4}$
$e_{c0} = 1.145$	$m_R = 6$
$e_{i0} = 1.317$	$m_T = 2$
$\varphi_c = 29.5°$	$\beta_\gamma = 0.2$
$h_s = 72$ MPa	$\chi = 1.0$
$n = 0.448$	
$\alpha = 0.12$	
$\beta = 3.0$	

Numerical element tests with the set of material parameters given above yielded the results shown in Figure 2 for an oedometric compression test with several unloading/reloading cycles, in Figure 3 (top) for a drained triaxial compression test with an initially very loose specimen and in Figure 3 (bottom) for a drained triaxial compression tests with an initially medium dense to dense specimen.

It is concluded, that the applied model with the given set of parameters enables a realistic modelling of the mechanical behavior of the granular material. For practical purposes it is interesting to note, that only conventional tests, which can be performed in every qualified geotechnical laboratory, are sufficient for a successful calibration process. There are no special requirements with respect to accuracy or experimental equipment.

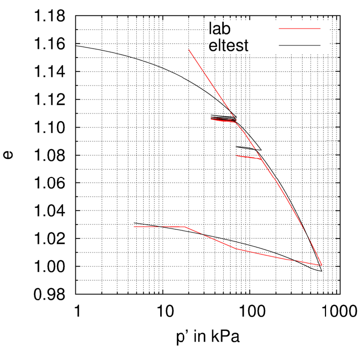

Figure 2. Oedometric compression with unloading/reloading cylces: actual test (lab) and numerical element test (eltest).

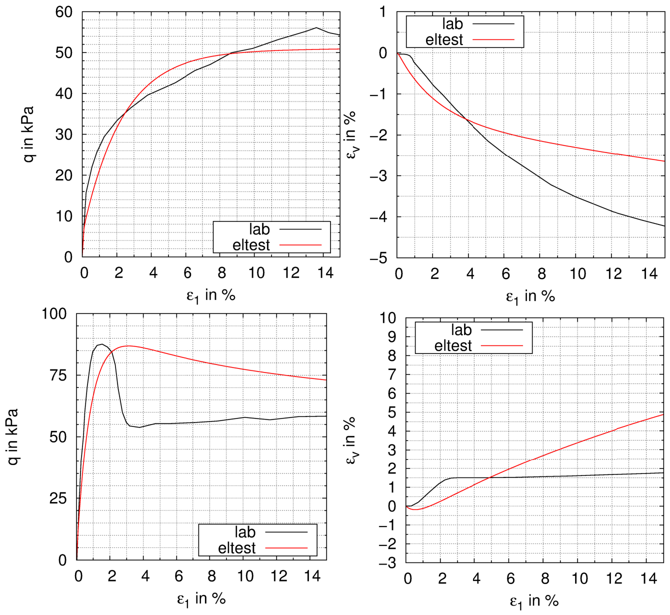

Figure 3. Triaxial compression (top: loose initial state, bottom: dense initial state): deviatoric stress vs. axial strain (left), volumetric strain vs. axial strain (right).

4 NUMERICAL MODELLING

4.1 *Outline*

In the following subsection the general performance of the applied constitutive model for alternating (cyclic) loading shall be demonstrated on the element level.

The subsequent subsection covers dynamic 3D finite element analyses of the field tests and a first study of a building founded on micro-pile groups in loose saturated sand exposed to seismic excitation.

4.2 *Cyclic element tests*

Numerical cyclic element tests under drained and undrained conditions were performed for the set of material parameters given in Section 3.2.

Figure 4 shows the degradation of the normalized (secant) shear modulus with increasing shear strain amplitude for drained conditions and the corresponding increase of the damping ratio . For the same isotropic initial stress state $p_0' = 50$ kPa, two results for the upper and lower bound initial void ratio are shown.

It is concluded, that the applied model is able to realistically describe the mechanical behavior of granular material exposed to cyclic loading under drained and undrained conditions, respectively.

It is known that the presented model tends to overestimate densification due to cyclic shearing under drained conditions and thus the generation of excess pore water pressures for undrained conditions. In case of only a few cycles (< 100), which is the case for e.g. earthquakes one can expect realistic results on the safe side with respect to possible liquefaction.

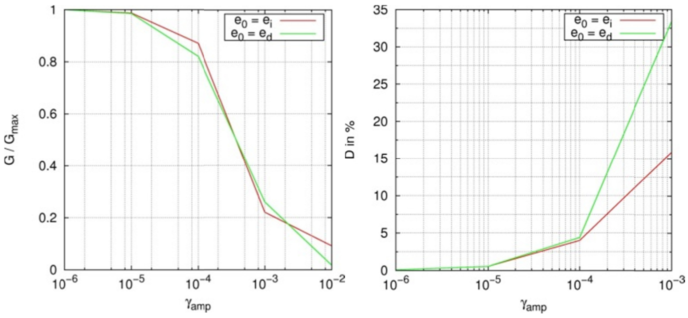

Figure 4. Decrease of normalized shear modulus with increasing shear strain amplitude (left), Increase of damping ratio D with increasing shear strain amplitude (right) under cyclic shearing.

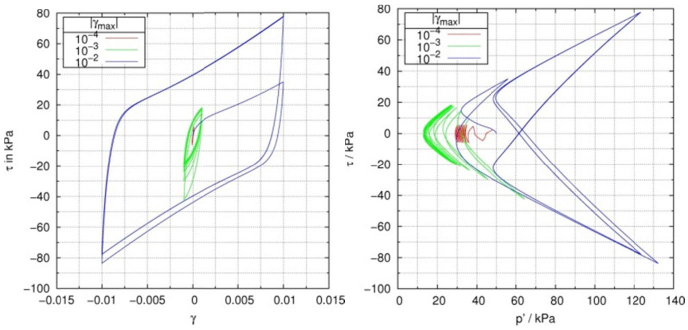

Figure 5. Shear stress τ vs. shear strain γ for undrained cyclic shearing (left), shear stress τ vs. effective mean pressure p' for undrained cyclic shearing (right) for different shear strain amplitudes.

Figure 5 shows the calculation results of undrained simple shear tests for three different shear strain amplitudes with 10 cycles each (p_0' = 50 kPa, e_0 = 0.7, dense state) and the corresponding change in the effective mean pressure p' in the τ-p'-diagram.

4.3 *Boundary Value Problems*

4.3.1 *Field Tests*

4.3.1.1 Model description

Two different models were created for the back analyses of the field tests. The first one considers the test pit filled with sand as carried out in situ together with the two test foundations with three piles each (assumed diameter: 30 cm). The heap of excavated earth next to the pit is also considered (Fig. 6). The second model is simplified to save computational costs in case it yields the same results as the more complex one. It contains only half of one test foundation and does not allow for the complete geometry of the test pit filled with sand and the heap of excavated.

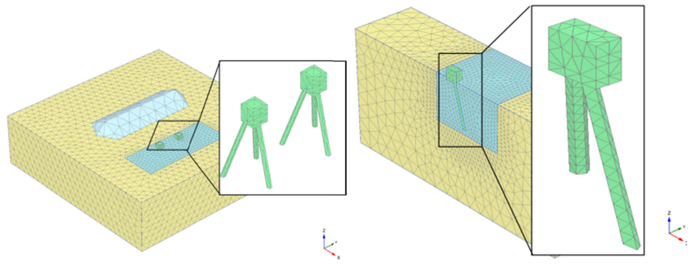

Figure 6. Complete and simplified 3D-FE model of the field test setup.

In both models absorbent boundaries are applied to the vertical edges and the base to prevent the reflection of mechanical waves during the dynamic excitation of the test foundations. The static vertical load of 1000 kN is applied as a uniformly distributed surface load (0.6 m x 0.6 m). The dynamic excitation is realized by giving prescribed velocities of the pile caps, as actually measured during the field tests on top of the pile caps.

Different mesh finesses were examined for both models, the same holds true for the maximum allowable time steps to prevent a negative influence on the calculation results. This is of special importance when using strongly non-linear constitutive models, such as hypoplasticity for dynamic analyses.

At this stage of the study interfaces between the piles and the adjacent soil are not allowed for. Experience from comparable 3D finite element analyses has shown that in case of relatively small displacements and sufficiently fine discretized piles one can obtain very similar result with and without interfaces. Nevertheless, this will be examined in the course of this ongoing project.

The structural elements (micro-piles and pile cap) as well as the soil surrounding the sand filled test pit are modelled as materials with linear-elastic behaviour, for the sand the hypoplastic model and the parameters mentioned above are used.

Geometric nonlinearity was not accounted for since large displacements were neither measured in the field nor expected in the numerical analyses.

4.3.1.2 Results

The following quantities were evaluated: Vertical displacement of pile cap due to static vertical loading with 1000 kN, vibration velocities of four points in the vicinity of the test foundation, vertical displacements of the pile cap due to dynamic horizontal loading.

The measured vertical displacements of the pile caps due to static vertical loading of 1000 kN were 3.1 mm and 4.9 mm, respectively. The difference can be explained by the fact, that the piles of the foundation with the higher settlement were constructed using a four-times lower injection pressure, which resulted in a smaller diameter of the piles and therewith a softer behavior when subjected to (vertical) loads (cf. Fig. 8).

The FE analysis with the simplified model yielded 7.5 mm, the one considering the complete setup 5.1 mm. The agreement with the measurements is satisfactory, especially taking into account the uncertainty of the resulting pile diameters due to the installation procedure. On the other hand the difference of the two calculations suggests, that there is still an influence of the spatial discretization of the two models.

The results of the vibration velocity measurements are depicted in Figure 7 for the first 4 seconds of a test run with $F = \pm$ 150 kN and a frequency f = 5 Hz. The picture in the upper left corner depicts the location of the measurement points. Besides Geophone G10, which was installed on top of the pile cap of the test foundation, all other geophones were installed 30 cm below the ground level. The agreement of the results is satisfactory.

The agreement of the numerical results with respect to the vertical displacements of the pile caps due to dynamic horizontal loading under constant vertical load is very poor, which is why the figures are omitted here. The calculated settlements were up to 50 times higher than the measured ones. The most striking reason for this discrepancy is illustrated in Figure 13. Successful dynamic tests in situ could only be performed with the front pile group. This was the one with the micro-piles produced with a higher injection pressure than conventionally applied, which resulted in piles with diameters up to approximately 60 cm. The assumption of 30 cm was only valid for the piles constructed with the lower (standard) injection pressure (rear foundation in Fig. 8). Figure 8 also suggests that the anchor piles could have an influence on the test results and should therefore also be considered in the model. The pile diameter and the anchor piles will be considered in the course of the project.

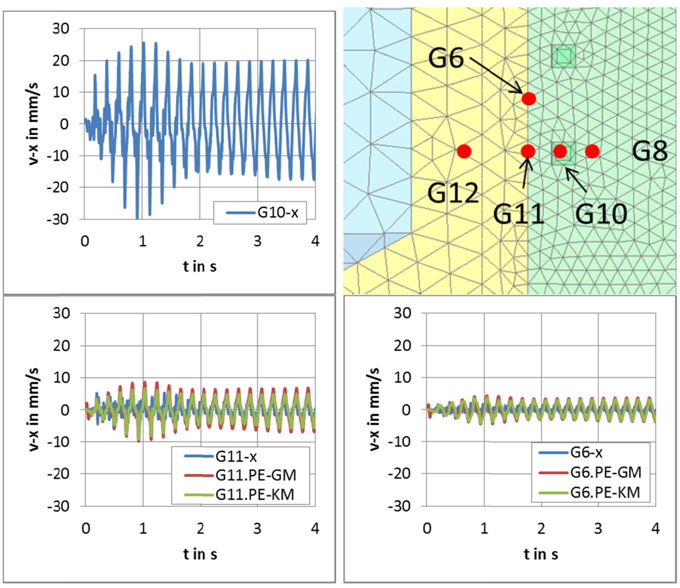

Figure 7. Measured and calculated vibration velocities.

Figure 8. Excavated test foundations after the field tests.

4.3.2 *Deep Founded Building exposed to Earth Quake in Liquefiable Soil*

4.3.2.1 Model description

Figure 9 shows the model of a building founded on four micro-pile groups with 4 micro-piles each. To safe computational costs, again only half of the building is modelled using symmetry in the (x-z)-plane.

Absorbent boundaries are applied at the vertical edges of the model, not including the plane of symmetry (y = 0, front face in Fig. 9).

The subsoil consists of two layers. Above a linear elastic (“rock”) base there is a 6 m thick layer of very loose water saturated sand (hypoplastic model and parameters as described in Sec. 3).

For the structural elements (micro-piles, base plate and building) linear-elastic behavior is assumed.

The earthquake excitation was realized by means of prescribed horizontal and vertical accelerations at the model base with data of the 2011 Christchurch earthquake.

Dynamic analyses under undrained conditions were performed to examine the influence of the micro-pile foundations on the generation of excess pore water pressures in the vicinity of the building during an earthquake.

4.3.2.2 Results

This first attempt yields promising results with respect to mitigation of liquefaction. Figure 10 depicts the generated excess pore pressures due to earthquake excitation (base plate of the building in upper left corner). It can be seen that the highest values of approximately 60 kPa occur outside the area of the building. Even between the two pile groups under the base plate the excess pore water pressures are significantly reduced compared to the adjacent soil.

This illustrative example shows how with the aid of a validated model (ongoing work) different layouts of micro-pile foundations shall be examined for specific subsoil conditions to obtain earthquake-proof foundations especially with respect to possible liquefaction.

Since the FE software used for the analyses is not able to allow for pore water flow during dynamic analyses the calculated accumulated excess pore water pressures are overestimated, which yields results on the safe side with respect to liquefaction.

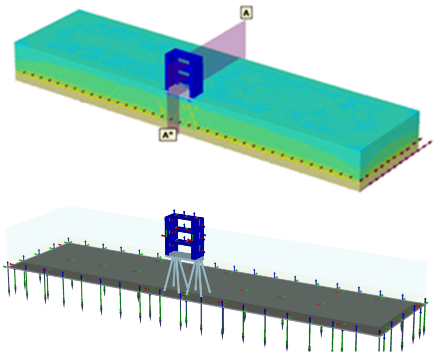

Figure 9. 3D-FE model for dynamic analyses of a building founded on micro-piles and exposed to earthquake excitation.

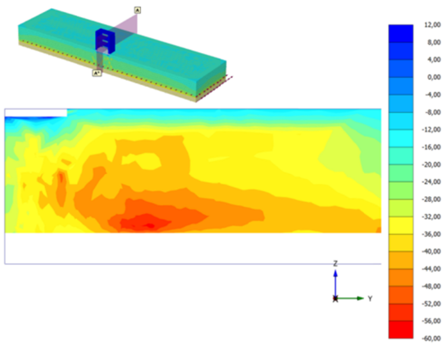

Figure 10. Excess pore water pressures during the earthquake for a cross-section through the middle of the building.

5 SUMMARY AND OUTLOOK

This paper describes full-scale field tests of micropile groups under static vertical load and exposed to dynamic horizontal excitation and the corresponding numerical analysis using hypoplasicity. The capacity of the applied hypoplastic model is demonstrated with the aid of actual and numerical element tests under drained and undrained conditions and for monotonic and alternating (cyclic) loading (Sec. 3 and 4).

The dynamic 3D finite element analyses of the field tests have not yielded satisfying results so far. This is mainly due to the fact the assumed geometry of the relevant test piles was underestimated (30 cm instead of up to 60 cm). The actual pile geometry as observed after the excavation of the test piles will be adopted in the further course of the project. The same holds true for a deeper examination of the spatial discretization of the model. The influence of interfaces between the piles and the soil will also be investigated. Additionally, the CPT results shall be used for an advanced interpretation using the Karlsruhe Interpretation Method (KIM) (Cudmani & Osinov 2001) for the determination of the initial void ratios.

Eventually, an illustrative example is given of how 3D dynamic Finite Element analyses shall be used in the course of the ongoing project to find efficient pile designs with respect to a mitigation of liquefaction in the adjacent subsoil.

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