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Performance of pile foundations and soil improvements in sensitive lacustrine clay by large-scale zone loading tests and loading tests on single elements

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ABSTRACT: Large deposits of lacustrine clay formed after the last glacial period around 11.000 years ago are commonly found today in regions close to the foothills of the Alps. Due to their sensitivity, characterised by a decrease in strength and stiffness when subjected to static or dynamic shear stresses, as well as the depth of their deposits, which in some cases reaches several hundred metres, these subsoil conditions present major challenges in foundation design and geotechnical engineering. Large-scale research under the acronym SEBRO is still ongoing and focuses on enhancing the efficiency of foundations in lacustrine clay. Utilising a unique, holistic research framework that incorporates state-of-the-art subsoil exploration techniques and monitoring systems, the study systematically investigates various methods of deep foundation and subsoil improvement. The experimental study of the foundations primarily consists of trial loadings involving a total of 27 tests on individual piles and soil improvement elements, as well as 12 zone loading tests on groups of piles and on improved soil, respectively, over a period of more than six months each. Due to the significant depth of the present deposit of sensitive lacustrine clay, which can reach up to 300 m, the focus of the investigations is on floating deep foundations and soil improvement. The time-dependent bearing behaviour of the trial foundations, with attention to installation effects, is therefore being extensively monitored and compared to a correspondingly loaded shallow foundation as a reference. Zone-loading tests involve a 5 m x 5 m foundation slab, subjected to a dead load of up to 6.25 MN, corresponding to a mean footing pressure of 250 kN/m². To study deformations related to consolidation and creep, the loading, unloading, and reloading phases last over several months. The initial results of this study indicate varying impacts on the soil structure, and therefore possible loss of shear strength and stiffness of the lacustrine clay, when considering construction-induced excess pore water pressures. Although the natural bearing capacity can be assumed to be affected by the installation processes, all the pile foundation methods and soil improvement techniques investigated so far provide significantly higher load-bearing capacities than the shallow foundation.

KEYWORDS: Sensitive soil, Lacustrine clay, Piles, Ground improvement, Large-scale trial loadings, Zone loading tests

1 INTRODUCTION

Lacustrine clays, a young Quaternary sediment commonly found close to the Alps in Austria, southern Germany, and Switzerland have a distinctive composition consisting primarily of low- to medium-plasticity clay, with silt and sand enclosed in sections ranging from a few millimetres to several centimetres in size. For natural, fine-grained soils, the state of the soil is affected by structural effects from their diagenesis and loading history, possibly due to viscous ageing, geometric orientation of the particles (fabric) and chemical bonding (Bjerrum and Lo 1963; Mitchell 1979; Leroueil and Vaughan 1990; Burland 1990; Wood 1991; Cotecchia and Chandler 2000). The microstructure of lacustrine clays is often metastable, which may result in significantly higher in-situ strength and stiffness than that measured in the laboratory on disturbed and reconstituted/remoulded samples, where the intrinsic soil state is governed by the stress and by the void ratio (Baracos et al. 1980; Scherzinger 1991; Krieg 2000; Kempfert and Soumaya 2004; Messerklinger and Springman 2010; Oberhollenzer et al. 2024). These microstructural bonds are inherently unstable and prone to collapse, particularly under shear stresses induced by static or dynamic loading. Consequently, changes in stress due to e.g. mechanical loading can cause varying degrees of disturbance to the structure, leading to a decrease in the soil’s shear strength and compressibility.

Based on the experience of challenging construction projects within the so-called Rosenheim basin, where a large layer of lacustrine clay forms a deposit of up to 300 metres deep, major scientific and practical questions have arisen about the mechanical behaviour of lacustrine clay, specifically Rosenheim lacustrine clay. These questions concern dealing with the structural loss (Cudmani et al. 2022; Rebstock et al. 2022; Rebstock et al. 2024).

- Which subsoil investigation strategies, comprising in-situ and laboratory tests, best characterise the mechanical behaviour, with a focus on the sensitive structure of lacustrine clay?
- To what extent do different techniques for constructing pile foundations and soil improvements reduce the natural stiffness and strength of the lacustrine clay?
- Can construction-induced deformation be qualitatively predicted, and possibly minimised or avoided?
- What is the performance of different pile foundations and soil improvement techniques, considering their resource and monetary demands and the complexity of their construction?

A five-year holistic research project under the acronym SEBRO was granted to study these questions, and an approximately 6,000 m² test site was rented in Kolbermoor, a municipality in the Rosenheim basin in southern Germany. Comprehensive subsoil investigations and various trial loadings are carried out at this site. Berz (2024) outlines the general research concept, in which 19 partners from the construction industry are involved, installing various foundations for testing, as well as providing services such as crane operation, material handling, earthworks, monitoring and subsoil investigations.

Due to the ongoing trial loadings and project-specific restrictions agreed with industry partners, results are currently presented without linking them to specific construction methods. After the on-site investigations are completed, as expected by the end of 2026, and the measurements have been evaluated, comprehensive publications will be produced to present the data collection.

2 SUBSOIL CHARACTERISATZION

The Kolbermoor test site is located within the Rosenheim basin, which lies within the range of the catchment area of the River Inn. The topography was formed by a large downhill-moving glacier during the last ice age and is characterised by glacial erosion of the formerly present subsoil, which consists of Miocene-age sediments. When the last ice age ended around

11,000 years ago, the glacier left behind a basin that formed the former Rosenheimer Lake. This lake covered an area of more than 420 km² (Kroemer 2011). Over time, the Inn River transported sediments, which filled the lake basin, depositing fine-grained soils, including silt and clay, in alternating layers. During flooding events, at least some fine sand was also transported into the lake. Sedimentation occurred through mechanical, chemical and biogenic processes (Schumann 1969). According to geophysical studies, these deposits are estimated to be at least 80 m thick at the Kolbermoor test site and may reach up to 300 m within the Rosenheim basin (Wolff 1973).

To investigate the subsoil conditions for soil classification and geotechnical characterisation at the Kolbermoor test site, various sampling methods and numerous field and laboratory tests were employed. The results of the trial loadings are currently being evaluated, but a more detailed comparison of the field tests, including various pressuremeter techniques and shear wave velocity measurements, has already been published (see Miraei et al. 2025). As presented in this publication, fundamental characteristics were determined by cone penetration tests with pore pressure measurements (CPTu).

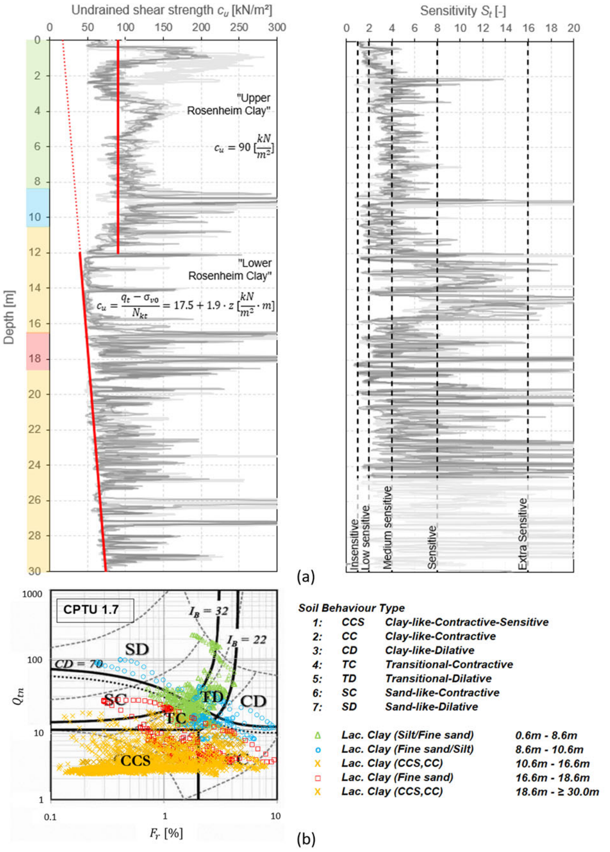

Figure 1. CPTu results: (a) derived c_u and S_t, (b) SBT evaluation based on (Robertson 2016).

Figure 1 (a) shows the undrained shear strength (c_u) and sensitivity (S_t) profiles derived from seven CPTu tests across the test site. In the shallow layers, labelled as "Upper Rosenheim Lacustrine Clay", there is no depth-dependent increase of c_u. Here, a general mean value of $c_u = 90$ kN/m^2 was determined; however, significantly larger results of up to $c_u = 300$ kN/m^2 were found, mainly in layers with a locally dominant fine sand fraction. At a depth of more than 10 m below ground level (bgl.), strong fluctuations in undrained shear strength are still present, with values of up to $c_u = 300$ kN/m^2. In contrast to the upper 10 m of soil, the lower limit of c_u increases with depth (z [m]), as given by the empirical equation $c_u = 17.5 + 1.9 \cdot z$, which indicates the presence of the so-called "Lower Rosenheim Lacustrine Clay".

Figure 1 (b) illustrates the classification from CPTu according to Robertson's (2016) soil behaviour type (SBT), confirming the presence of sensitive soil layers (CC and CCS) in the Lower Rosenheim Lacustrine Clay. The high values of c_u at approximately 9 m bgl. and 18 m bgl. correlate to comparably high permeability, as measured via dissipation tests (CPTu), and the results of Marchetti Dilatometer (DMT) and Self-Boring Pressuremeter (SBPM) tests, partly shown in Miraei et al. (2025), suggesting layers with increased fine sand content.

3 BOUNDARY CONDITIONS AND SPECIFICATIONS OF THE TRIAL-LOADINGS

Zone loading tests are performed on all 12 of the investigated foundation techniques. Additionally, if technically feasible and applicable, three individual elements are constructed for compression load tests for each pile foundation and soil improvement technique. Figure 2 illustrates the general layout of the Kolbermoor test site.

The approximately 0.9 m thick layer of topsoil and peat originally present was replaced by a 0.6 m thick geosynthetic-reinforced working platform. To prevent unwanted load transfer through the working platform during loading tests on individual elements, the geosynthetic reinforcement is cut around the piles and soil improvement elements. The working platform's friction is also excluded. No geosynthetic reinforcement is present in the zone loading tests.

As shown in Figure 2 a regular grid comprising 222 prefabricated vertical drains (PVDs) with a total length of 3,774 m has been installed. The PVDs ensure homogenised drainage conditions and accelerated consolidation, while limiting the effects from excessive pore pressure build-up on neighbouring test fields during the construction of the trial foundations.

The methods for pile foundations and soil improvement are investigated in terms of soil-structure interaction and the various degrees of disturbance to the naturally occurring structure of the lacustrine clay induced by construction. The extent of the effects of a group of piles or individual soil improvement elements differs from that of the construction of a single pile or soil improvement element. To clearly identify these influences, and in correspondence with the geotechnical boundary value problem of real building foundations, the zone loading tests were planned.

In respect of the potential dead load, material handling and costs, as well as the inclusion of a variety of piling and soil improvement techniques, the foundation depth has been chosen to be 12.6 m bgl., where the expected soil strength and stiffness are low, to emphasise the investigation of floating foundations. Two soil improvement methods are technically limited to a construction depth of less than 12.6 m but have been chosen for examination due to their widespread presence in the market and technical innovation, respectively.

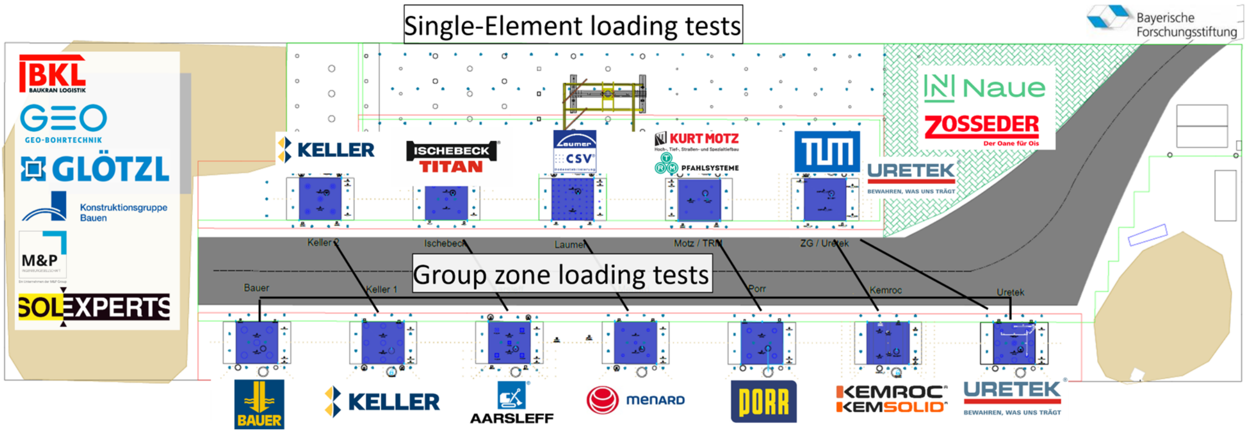

Figure 2. Layout of the Kolbermoor test site for the loading tests on individual elements and zone loading tests (marked as purple squares) and PVD (blue dots) including the individual company logos of the partners within the SEBRO research.

Table 1. Overview of the pile foundations and soil improvement methods examined by zone loading tests.

Executing company	Foundation	Dimensions [cm]	Length [m]	Quantity [-]
Aarsleff Spezialtiefbau GmbH	*Precast Reinforced Concrete Piles, type Centrum Pfahl*	40 x 40	12.6	5
Bauer Spezialtiefbau GmbH	*Vibrated compacted granular columns (via vibratory hammer)*	Ø 75 *	t.b.d.*	t.b.d.*
Friedr. Ischebeck GmbH	*Drilled Micropiles, type TITAN*	Ø 20	12.6	9
Keller Grundbau GmbH	*Vibrated compacted granular columns (via deep vibratory compaction)*	Ø 60 *	t.b.d.*	t.b.d.*
Keller Grundbau GmbH	*Vibro Mortar Columns*	Ø 60 *	t.b.d.*	t.b.d.*
Kemroc Spezialmaschinen GmbH	*FMI-Method, type KSI*	500 x 50	10.2	3
Kurt Motz […] GmbH	*Ductile Driven Piles, type TRM*	Ø 27	12.6	5
Laumer CSV […] GmbH	*Dry Mortar Columns, type CSV*	Ø 19	5.0 – 6.9	57
Menard GmbH	*Wet Mortar Columns, type CMC®*	Ø 32	12.6	5
Porr Spezialtiefbau GmbH	*Screw Piles, type Atlas Piles*	Ø 46/56	12.6	4
Uretek Deutschland GmbH	*Hybrid Injection and Deep Injection*	Ø 30 *	t.b.d.*	9*
Technical University of Munich	*Shallow foundation (Reference)*	-	-	-

*to be determined (31.07.2025)

4 ZONE LOADING TESTS

4.1 *General considerations and methodology*

This research project is distinguished by the large-scale test loads applied to group elements using 12 different foundation methods (including a shallow foundation as a reference, see Table 1), all within a single test site with quite homogeneous subsoil conditions. The methods include various pile and soil improvement methods, as well as different construction methods, such as (leader-mounted) driving, jacking, injecting and vibrating, and different materials, such as concrete and gravel. For a given foundation depth and element spacing, the quantity of elements varies as listed in Table 1.

To qualitatively compare the performance of the foundation methods listed in Table 1, geotechnical and geodetic monitoring, as shown in Figure 3, is implemented for all test fields. Preliminary simulations using the Finite Element Method (FEM) and project-related experiences have led to the selection of locations and suitable depths for the monitoring sensors and measurement pipes in each test field:

- 3x pore water pressure sensors (PWD)
- 2x comb. earth- / pore water pressure sensors (ED/PWD)
- 2x inclinometer tubes (INKL)
- 1x combined sliding deformeter / inclinometer tube (GD/INKL)
- 2x extensometer (1x either 3 or 5 anchoring points, EXT)
- 4x hydrostatic gauge sensors (HGS)
- >90 geodetic measurement points (GMP).

The comprehensive monitoring system enables continuous recording of settlement (HGS), compressive strain (EXT) and pressure (ED/PWD). GD/INKL and GMP are measured manually at specific intervals. In addition, changes in soil properties are recorded using seismic cross-hole (CH) measurements from the INKL tubes, where the shear wave velocity (v_s) is evaluated. In addition to focusing on ground deformation during construction, numerous measurements are taken during loading, unloading, and reloading phases. Specific foundation elements are partly instrumented with distributed fibre optic sensors and rebar strain gauges.

Four identical 32-ton rafts of reinforced concrete were constructed for stacking the dead loads, which can be moved by a mobile crane to the individual fields of the zone loading tests. As the primary objective of the research is to examine soil-structure interaction regarding the lacustrine clay, the foundation element heads are cut as precisely as possible at one level. This ensures that, in the context of a combined pile-raft foundation, the load is primarily transferred from the raft to the foundation elements, minimising the soil loading from the raft between the elements. Initially, finite element analyses (FEA) were performed to model the constitutive behaviour of the lacustrine clay, incorporating a simplified pressure-dependent elastic stiffness and ideal plasticity. These analyses predicted a pile-raft coefficient $\alpha_{pr} = \sum R_{pile}/R_{tot}$ between 0.4 and 0.6, depending on the diameter and quantity of the foundation elements beneath the raft. This clearly needs to be validated through back calculations after evaluating the monitoring data, considering the total R_{tot} and pile-specific mobilised resistance R_{pile}.

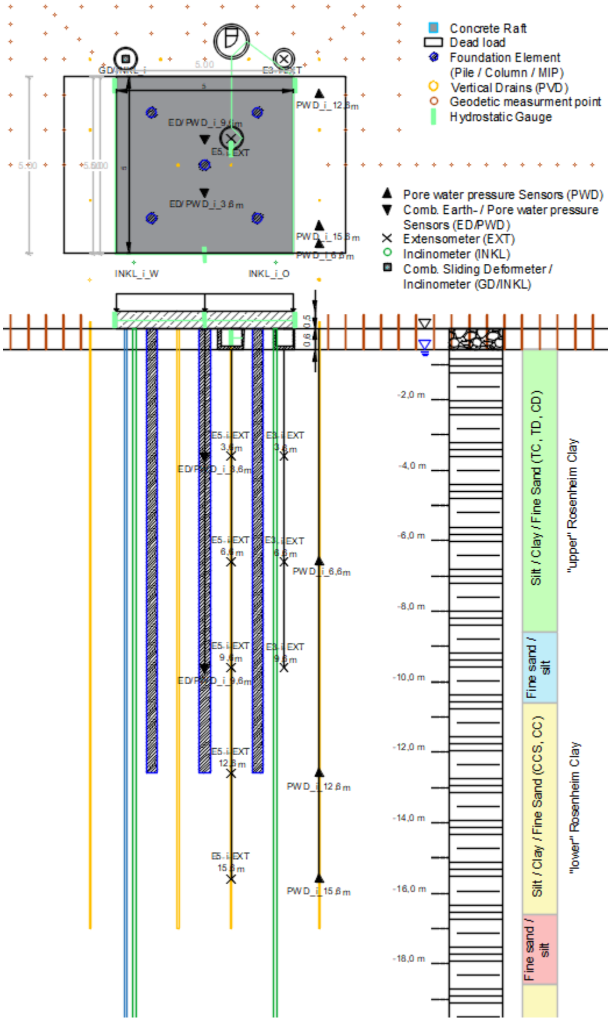

Figure 3. Monitoring of the zone loading tests.

For all zone loading tests, dead load increments of approx. 2.1 MN are applied to simulate the construction process and monitor time-dependent deformation due to consolidation and creep. At the maximum loading step 3 (Lst. 3), the load of approx. 6.25 MN is held constant for at least three months. Including unloading from loading step 2 to loading step 1 and reloading, each zone loading test takes about six months to complete loading step 3. After this, the dead loads are transferred to the adjacent zone loading tests.

Construction of the foundations for the zone loading tests takes place approx. one month prior to the trial loading. During construction at adjacent test fields, deformations and pressure changes are recorded and evaluated to assess the impact of various piling and soil improvement methods on existing structures.

4.2 *Results from loading the shallow foundation*

The shallow foundation is used to study soil-structure interaction, independent of the impact of pile construction or soil improvement. For shallow foundation construction and thus its load-bearing capacity, it is assumed that the disturbance to the soil structure is minimal. Comparably soft vertical elements were placed through the working platform right at the boundaries of the foundation slab to create an artificial shear zone and ensure full load transfer to the lacustrine clay.

Figure 4 shows the situation when the maximum dead load is applied to test fields 1 and 2 by stacking precast reinforced concrete piles reaching a height of about 10 m.

Figure 4. Group zone load tests including shallow foundation are shown in loading step 3.

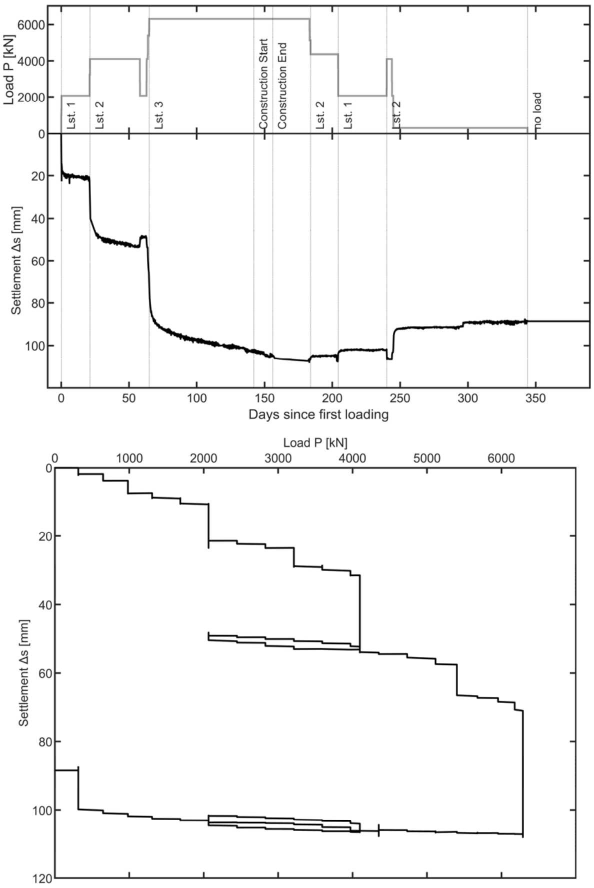

Figure 5. Loading sequence and measurements (a) settlement Δs against time and (b) settlement against load P for the shallow foundation, represented by the mean value of the HSGs.

Figure 5 (a) visualises the loading sequence and the corresponding settlement over time. At around 140 days after the start of the loading sequence for the shallow foundation, construction of the two adjacent test fields began. Despite distances of 10 m or more, an increase in the settlement rate was observed, attributed to the two deep foundation methods employed.

4.3 *Results from loading pile foundations and soil improvement methods*

Figure 6 shows the results on the ongoing research into permanent changes in the total horizontal stress $\Delta\sigma_{max}$ at a depth of 9.6 m, alongside peak excess pore water pressures Δu_{max} during construction of the first seven constructions and zone loading tests. The PVDs ensure almost complete dissipation of excess pore water pressure within approx.. 48 h, but during some constructions, a temporary loss of effective stresses and thus at least local liquefaction of the soil is evident.

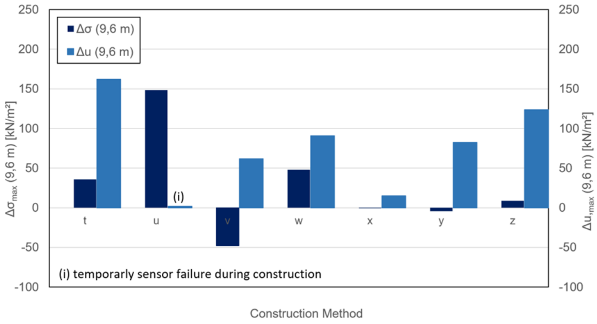

Figure 6. Permanent change of total horizontal earth pressure and temporary excess pore water pressure at 9.6. m bgl. due to construction.

Figure 7 shows the curves of the applied mean stress versus normalised settlement (i.e. s/s_{max}), for three foundation techniques. The different behaviour of the systems, particularly regarding creep deformation, may be related to the degree of soil disturbance during installation, but it may also depend on the varying number and diameter of the piles and soil improvement elements, respectively. A comprehensive analysis and comparison of system performance requires all influencing factors to be consistently considered.

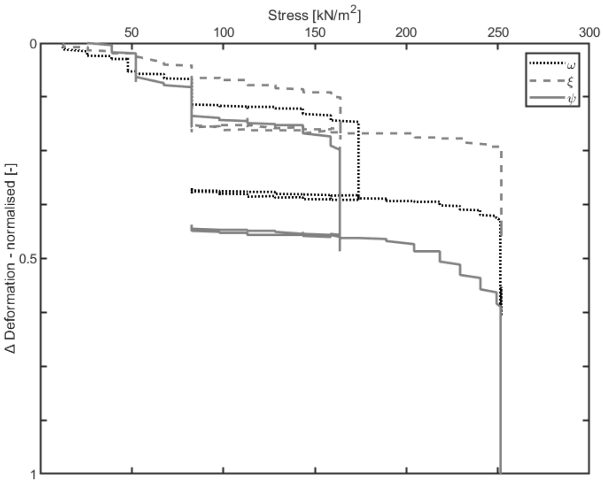

Figure 7. Applied mean stress from the loading against the normalised settlement, with examples shown for three improvement methods (here labelled: “ω”, “ξ”, “ψ”)

5 LOADING TESTS ON INDIVIDUAL ELEMENTS

In practice, load tests are typically performed on individual piles or soil improvement elements. This usually enhances the efficiency of foundation design by assessing load-settlement behaviour and bearing capacity. In many cases, these tests involve applying a tensile load, which is less labour-intensive than tests involving compressive loadings. However, if tensile load tests are conducted, the contribution of the base resistance cannot be measured and is therefore disregarded in the geotechnical design. However, for short piles and soil improvement elements, respectively, base resistance may play a significant role in load transfer and should therefore not be disregarded when aiming for an efficient foundation design. For this reason, it was decided to test the individual elements under compression. At least two out of three loading tests examined identical diameters and lengths, which correspond to the geometry of the piles and soil improvement elements, respectively, as constructed for the zone loading tests. The load-bearing capacity, and therefore the ultimate limit states (ULS), of the different single elements are estimated to lie between 0.6 MN and 2.0 MN. Differences in load-bearing capacity may be influenced not only by the different installation techniques, such as displacement and vibration, which correspond to varying degrees of soil disturbance. They may also be influenced by variations in cross-sectional dimensions, which range from 0.028 m^2 to approximately 0.44 m^2

The testing procedure is based on DIN EN ISO 22477-1 and involves applying static load increments of $\Delta F_c = 20$ kN, 80 kN and 160 kN, respectively (incremental loading, IL) in accordance with the expected bearing capacity. The duration of the creep phases was adjusted to limit the pile head settlement rate to < 0.1 mm / 20 min. In addition to stepwise loading and measurement of the creep coefficient (α), constant-rate-of-penetration (CRP) testing is conducted in the second phase of each loading test. A load relief occurs during overnight pausing before proceeding with at least two CRP phases, which include several abrupt changes in penetration rate, to evaluate the portion of the viscous resistance.

Considering the isotachs, as shown in Figure 8, which displays the different penetration rates during the CRP, the viscosity index (I_v) can be determined based on Krieg’s (2000) descriptions. Based on the current state of data evaluation from 12 out of 27 loading tests on individual elements, I_v ranges between 0.024 and 0.031, with a median of 0.028.

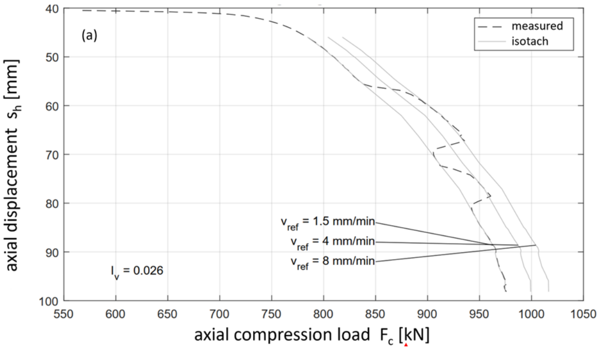

Figure 8. Exemplary load-displacement curve from a loading test on an individual element.

Although the subsoil conditions are homogeneous and the geometry of the elements and test conditions remain unchanged, significant variations in bearing capacity can be observed. So far, almost all load tests on individual elements have reached the ultimate limit state (ULS), as defined by the German Recommendations on Piling (EA Pfähle 2012). This is due to settlements exceeding 0.1 times the diameter (D) of the elements.

6 CONCLUSIONS

This comprehensive research project aims to answer key questions and improve the efficient and therefore sustainable design of foundations in sensitive lacustrine clay. The investigations specifically consider the influence of foundation construction and installation effects on bearing behaviour, using trial loadings on individual elements and zone loading tests on groups of piles, as well as soil improvement techniques, respectively. Preliminary results confirm that the bearing capacity depends on construction-induced soil disturbance, which can significantly affect the efficiency of foundation techniques in sensitive lacustrine clays. As of mid-2025, eight out of twelve foundation techniques have been constructed, and six have been tested.

The trial-loading results from the shallow foundation tests confirm that the overall experimental concept, specifically the monitoring system, was well designed. These results serve as a reference for evaluating the behaviour of pile foundations and soil improvement methods compared to the bearing behaviour of relatively undisturbed sensitive lacustrine clay. The chosen instrumentation enables detailed documentation of changes to the soil structure during foundation construction and trial-loading.

The foundation techniques that have been investigated so far are characterised primarily by fully soil-displacing piles and soil improvement elements. Despite the earth pressure sensors being placed at a distance of approximately twice the pile diameter, only a moderate increase in horizontal stress has been observed. By contrast, most constructions so far have generated significant excess pore water pressures, indicating substantial soil disturbance, in some cases approaching local liquefaction. This occurred despite the strict limitation of the construction rate and the initial installation of prefabricated vertical drains spaced 2 m apart to facilitate relatively rapid pore pressure dissipation.

Based on the results of zone loading tests and corresponding loading tests on individual elements, comparative numerical simulations and comprehensive analyses will be conducted to determine the most efficient application range of the individual foundation techniques and to further improve the construction technologies.

7 ACKNOWLEDGEMENTS

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8 REFERENCES

Berz, Patrick (2024). Effizientes und nachhaltiges Bauen auf strukturempfindlichem gering tragfähigem Untergrund - SEBRO. In: Deutsche Gesellschaft für Geotechnik e.V. (Ed.). 38. Baugrundtagung Tagungsband-Spezialsitzungen 2024. Essen, Deutsche Gesellschaft für Geotechnik e.V. (DGGT), 23–30.

Bjerrum, Laurits/Lo, K. Y. (1963). Effect of Aging of the Shear-Strength Properties of a Normally Consolidated Clay. Géotechnique 13 (2), 147–157. https://doi.org/10.1680/geot.1963.13.2.147.

Cudmani, R./Rebstock, D./Schorr, J. (2022). Herstellung und Tragverhalten von Mischgründungen im Seeton: WT Rosenheim und SEBRO. In: Deutsche Gesellschaft für Geotechnik e.V. (Ed.). 37. Baugrundtagung. Vorträge: RheinMain CongressCenter Wiesbaden 05.-08.10.2022. Essen, Deutsche Gesellschaft für Geotechnik e.V. (DGGT), 317–327. Available online at https://www.dggt.de/index.php?option=com_content&view=article&id=388&Itemid=172 (accessed 12/28/2023).

Deutsche Gesellschaft für Geotechnik e.V. (Ed.) (2012). Empfehlungen des Arbeitskreises "Pfähle". EA-Pfähle. 2nd ed. Berlin 2012.

DIN EN ISO 22477-1. Stat. axiale PPB auf Druck, Dezember 2019. Berlin.

Krieg, Stefan (2000). Viskoses Bodenverhalten von Mudden Seeton und Klei. Dissertation. 150. Karlsruhe, Universität Fridericiana.

Kroemer, Ernst (2011). Extent of the late glacial lake Rosenheim and implications of isostatic movements Innsbruck, 2011.

Miraei, Mohsen/Nguyen, Ba-Phu/Cudmani, Roberto/Berz, Patrick/Csuka, Antal/Vogt, Stefan (2025). Comparison of Various Pressuremeter Tests for Characterizing Sensitive Lacustrine Clays. In: Proceedings of the 8th International Symposium on Pressuremeters, Luxembourg, 2 - 5 September 2025.

Rebstock, D./Cudmani, R./Vogt, S. (2024). (Misch-)Gründungen im Seeton - Erfahrungen in Rosenheim und aktuelle Forschung. In: 14. Österreichische Geotechniktagung -. Vorträge: Gründungen, Wien, 01.-02.02.2024.

Rebstock, D./Schorr, J./Cudmani, R./Kergl, K. (2022). Beurteilung der Messergebnisse der Gründungen einer Schrägseilbrücke in Rosenheimer Seeton. In: 36. Christian Veder Kolloquium. Vorträge: Bauen in weichen Böden, Graz, 28.-29.06.2022, 227–244.

Robertson, P. K. (2016). Cone penetration test (CPT)-based soil behaviour type (SBT) classification system — an update. Canadian Geotechnical Journal 53 (12), 1910–1927. https://doi.org/10.1139/cgj-2016-0044.

Schumann, Walter (1969). Geochronologische Studien in Oberbayern auf der Grundlage von Bändertonen. München, Verlag der Bayerischen Akademie der Wissenschaften.

Wolff, Hans (1973). Geologische Karte von Bayern 1:25.000 - Erläuterungen zum Blatt Nr. 8238 Neubeuern. München, Bayerisches Geologisches Landesamt.

Wood, David Muir (1991). Soil Behaviour and Critical State Soil Mechanics. Cambridge University Press.