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Back analysis of long-term measurements of a high-rise building founded on a raft foundation in overconsolidated clay

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ABSTRACT: In Frankfurt, Germany the high-rise building SGZ-Bank was constructed in the early 1970s with a raft foundation in the Frankfurt Formation which comprises mainly overconsolidated clay. During construction extensive measures were taken to prevent tilting of the building which however showed no long term improvement so that in 1980 maximum settlements of $s = 30.6$ cm and an angular distortion of $\alpha = 0.32\%$ were recorded. In the scope of this paper the results of a numerical simulation of the construction process of the SGZ-Bank are presented and compared to the measurements taken over a decade. In the 3D coupled pore pressure-displacement finite element analysis the visco-hypoplastic material model AVISA was applied to capture the time-dependent material behaviour of the overconsolidated clay.

Keywords: raft foundation; alternating loading; clay; visco-hypoplasticity; coupled pore pressure-displacement analysis

1 INTRODUCTION

Since the application of numerical methods in geotechnical engineering, the back analysis of measurements on case histories or model tests was carried out with aim of determining the initial stress state, the material parameters or the boundary conditions of a boundary value problem (Sakurai, 1990). In the last decades, there has been a rapid development of material models that allow for a realistic simulation of the stress-strain behaviour of soil. The validation of these material models is of crucial importance for their application in engineering practice.

Well-documented case histories such as the SGZ-Bank in Frankfurt, Germany (e.g. Leonhardt, 1972; Reul et al., 2007; Stahlmann et al., 2001) are particularly suitable for validating the functionality of material models. This paper focuses on the back-analysis of the available measurements on the SGZ-Bank which is founded in the highly plastic, overconsolidated clay of the Frankfurt Formation by means of three dimensional coupled pore pressure-displacement finite element analyses (FEA). The material behaviour of the Frankfurt Formation has been modelled with the material model AVISA (Tafili and Triantafyllidis, 2020) which allows to consider creep effects. The back-analysis of the partial demolition of the SGZ-Bank and reconstruction to the Park Tower as reported e.g. by Reul and Remmel (2009) will be the subject of a future publication.

2 CONSTRUCTION HISTORY OF THE SITE

The SGZ-Bank site is located at Bockenheimer Anlage 44 in Frankfurt am Main in vicinity of the old opera house. The building complex has undergone a number of changes up to the present day which have been documented by e.g. Leonhardt (1972), Stahlmann et al. (2001), Remmel et al. (2006), Reul et al. (2007), Reul and Remmel (2009). If not indicated otherwise, details on the various structures and construction processes presented in the following sections have been taken from Reul and Remmel (2009). Initially, the 96.6 m high SGZ-Bank with a 2-storey annex including an underground parking was constructed between 1970 and 1972. Between 2000 and 2001 the old SGZ-Bank annex was demolished and replaced by the 6-storey Atrium building. In the course of the Park Tower construction project from 2006 to 2007, the existing high-rise building was completely stripped, the facade structure was dismantled and the three top floors were demolished. The storeys that had been removed for structural reasons were reconstructed as a reinforced concrete skeleton structure and raised by three additional storeys. On the north-east side, a new 28-storey building was connected to the existing building. The various project stages are shown in Figure 1.

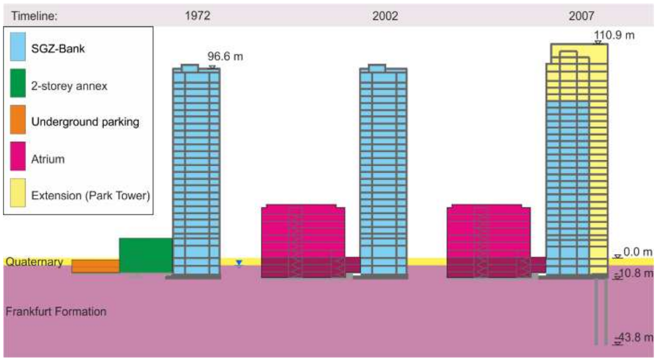

Figure 1. From the SGZ-Bank to the Park Tower

3 SGZ-BANK BUILDING

The structure of the SGZ-Bank high rise building consists of a reinforced-concrete skeleton construction with a curtain facade. The foundation was constructed as a shallow foundation with a 2.7 m thick raft at a depth of 10.8 m below ground level. The core was arranged asymmetrically on the north-west side of the raft (Figure 2). Using the slip-form construction method the core was built in advance and was almost completed within a month. Due to the asymmetry of the core, non-uniform settlements with an increasing tilting of the raft towards the north already occurred during construction (Leonhardt, 1972).

In the process of completing the shell of the building, the following measures shown in Figure 2 were taken to reduce tilting at various stages of construction:

- Undercutting of the high-rise raft along two strips at the south-western edge (connection to the 2-storey annex, U1) and south-eastern edge (basement extension, U2). According to Leonhardt (1972) the undercutting at the south-western edge closed as soon as March/April 1971.
- Creation of cavities by means of embedding the plates on polystyrene pads, which were subsequently dissolved by injecting a solvent (CHC).
- Removal of a 40 cm to 90 cm wide concrete block (W1) in the area of the basement extension. The presence of the concrete block, a remnant of the excavation pit wall, results in a 50 cm widening of the raft and thus in an additional eccentricity and back-turning moment.
- Application of additional dead weight in two steps (B1 = 9.2 MN / B2 = 4.8 MN) on the basement extension, which had been constructed as a cantilever beam.
- Part of the load of the 2-storey annex is applied as a line load at the south-western edge of the raft (L1).

The settlement-inducing structural load amounts to $P = G_{raft} + G + Q/3 = 285.6$ MN without consideration of uplift. The variation of the load with time is plotted in Figure 3, which also shows a reduction of the total effective load of approximately $\Delta P' = 38.8$ MN due to the rise of the groundwater level at the end of construction. The distinctive feature of the construction process is the load of 86.8 MN applied in the core area (Figure 3) right at the beginning within only one month, amounting to 30% of the total load. In the following 9 months, both the shell of the high-rise building and the 2-storey annex were completed. In the last year of construction, the load increases slowly to its final value, mainly due to the interior work and installation of the facade.

4 SUBSOIL CONDITIONS

Foundations in the city centre of Frankfurt show time-dependent load-deformation behaviour which is caused by Tertiary layers, including the Frankfurt Formation and the underlying Frankfurt Limestone. The Frankfurt Formation consists of an irregular sequence of clays and clay marls, silt and sand layers of varying thickness, limestone and dolomite banks. This results in the layer as a whole having an inhomogeneous appearance. In the older literature, the Frankfurt Formation was generally termed ‘‘Frankfurt Clay’’, e.g. Sommer et al. (1991) and Reul and Randolph (2003). However, in this paper the term Frankfurt clay will be used only for the clayey soils of this layer. The rocky Frankfurt Limestone, which is composed of massive limestone and dolomite layers, algal reefs, marly calcareous sands and silts, as well as marly clay, has a very low compressibility compared to the Frankfurt Formation and, thus, is not settlement-relevant. The groundwater circulates in the Quaternary sand and gravel as well as in the Tertiary sand and limestone bands, while the Tertiary clay is practically impermeable. Because the Quaternary and Tertiary aquifers are connected, a groundwater drawdown in the Tertiary layers may result in a reduction in the hydraulic head within an area with a radius of several hundred meters.

On the project site, the subsoil conditions are essentially characterised by a 5 m thick layer of artificially filled soils and Quaternary sand directly below the ground surface followed by a 64 m thick layer of the Frankfurt Formation underlain by the Frankfurt limestone. Ganal (2023) determined a total share of 17.56 % sand and limestone bands (sand: 12.53 %, limestone: 5.03 %) in the Frankfurt Formation at the site. The groundwater level lies directly below the bottom of the Quaternary sand at a depth of approximately 5.3 m.

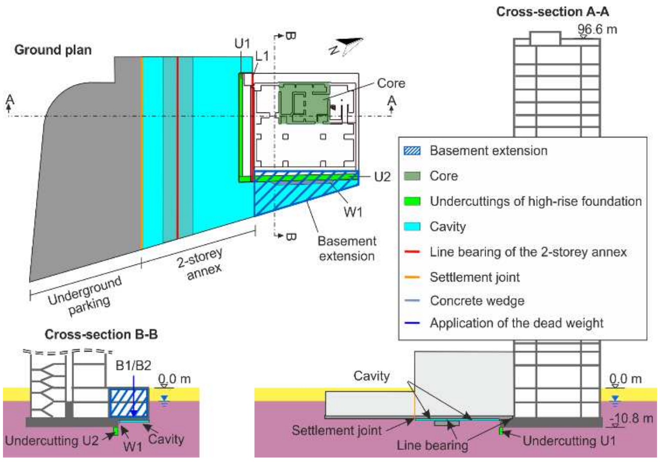

Figure 2. Measures to reduce settlement and tilting

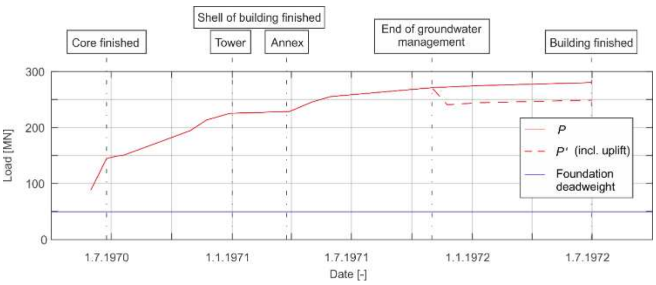

Figure 3. Variation of the settlement-inducing structural load with time after Stahlmann et al. (2001)

5 NUMERICAL ANALYSIS

5.1 Finite element model

The numerical study was carried out by means of 3D FEA with the code Tochnog (Tochnog Professional Company, 2022). With the coupled pore pressure-displacement analyses the time dependent displacement caused by consolidation processes as well as by the material behaviour of the soil is modelled. The FE mesh shown in Figure 4a comprises 22384 hexahedral elements with both displacements and pore pressures varying linearly across the elements. The boundary between the Frankfurt Formation and the Frankfurt limestone which represents the bottom of the FE mesh was assumed to be 58.6 m below the bottom of the raft. The ground area of the model amounts to 113.3 m × 117.6 m. Instead of modelling the soil above the base of the raft with elements, a distributed area load of 200.1 kPa was applied to take the weight of the soil into account. In the borehole logs a layer of approximately 16 m thickness was identified immediately below the raft, in which hardly any sand and limestone bands were found. In order to reproduce the behaviour of the foundation more accurately, for this upper layer indicated in Figure 4a the properties of the Frankfurt clay corresponding to the base values documented in Table 2 were considered in the FEA. For the lower layer the parameters for the Frankfurt Formation specified in Table 2 for the SGZ-Bank site were applied. The raft was modelled with a thickness of t_r = 2.7 m (Figure 4b). The raft of the basement extension, which has no contact to the soil and therefore behaves as a cantilever beam, has a thickness of $t_{r,Basement}$ = 0.7 m. The raft is considered to behave linear-elastically (Young's modulus E = 30000 MPa; Poisson's ratio ν = 0.2).

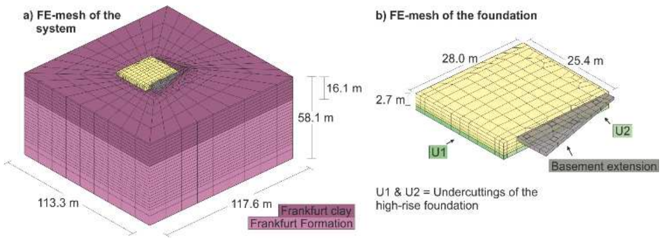

Figure 4. Finite element mesh

For the coupled displacement-pore pressure analysis it is assumed that the pore fluid flow is governed by Darcy's law and the fluid (water) is incompressible as well as the soil is fully saturated (S_r = 1). Drainage of the soil takes place beneath the raft as well as at every model surface. The raft is assumed to be impermeable. Table 1 summarizes the step-by-step analysis starting with the construction of the SGZ-Bank in 1970 until 2000, 28 years after the SGZ-Bank was finished. Excavations were modelled by reducing the load applied on the top edge of the model in the respective areas. The total dead weight of the raft including the raft of the basement extension of G_{raft} = 49.4 MN was modelled as a gravity load. The total settlement inducing load of the superstructure of $P = G + Q/3$ = 263.2 MN was applied on top of the raft. Details of the load evaluation and application are documented by Ganal (2023).

The groundwater level is controlled by a defined hydraulic pressure head at the drainage surfaces mentioned above. Groundwater level changes are then considered by changing the hydraulic pressure head.

In order to reproduce the behaviour of the foundation, all means to counter the tilting of the foundation described in section 3 were simulated in the FEA. Two element groups along two strips under the raft were defined (Figure 4b), to simulate the undercutting of the foundation. At the time of undercutting, the stiffness of the respective element groups was reduced to almost zero in order to prevent loads from being transferred. The additional dead weights (B1 & B2) on the basement extension and the load from the 2-storey extension are applied via a distributed surface load on the respective area.

Table 1. Step-by-step analysis of the construction process

Step	***P*** [MN]	**GW** [m.a.s.l.]	***Δt*** [*d*]	**Date**
1 In situ stress state	-	96.7	-	-
2 Start GW drawdown	-	92.3	19	10.02.70
3 Excavation to 10.7 m below ground level	-	91.2	28	01.03.70
4 Raft dead weight	49	91.2	6	16.05.70
5 Raft stiffness	49	91.2	2	22.05.70
6 Loading + B1 & B2	170+14*	91.2	102	24.05.70
7 Loading and U1	171+14*	91.2	1	05.09.70
8 Loading - B2	244+9*	91.2	210	06.09.70
9 Loading and U2	245+9*	91.2	1	03.04.71
10 Loading - B1	269	91.2	214	04.04.71
11 Loading + End of the GW drawdown	286	95.6	240	01.11.71
12 Consolidation / creep	286	95.6	10146	01.07.72

P total settlement inducing load at the respective time $P_{total} = G_{raft} + G + Q / 3$
GW ground water level in m above sea level
Δt duration of the process
Date starting date of the associated analysis step
* additional load on A10 because of B1 & B2

5.2 Material parameters and state variables for the Frankfurt Formation

The FEA on the load-bearing behaviour of the foundation of the SGZ-Bank are carried out with the AVISA model (Tafili and Triantafyllidis, 2020), which allows to simulate both creep and alternating loads and is therefore well suited for the current boundary value problem. The material parameters applied for the Frankfurt Formation are summarised in Table 2. The base values given for the Frankfurt Formation reflect the parameters of the Frankfurt clay derived from a large number lab tests with samples taken from different sites across the city and the back analyses of element tests (Ganal, 2023; Tafili et al., 2023) without considering embedded sand and limestone bands.

While the material parameters of the Frankfurt clay inside the Frankfurt Formation show not much scatter locally, the share of the much stiffer embedded sand and limestone bands varies from site to site. In order to capture the effect of the increased stiffness of the limestone and sand bands in the FEA, Ganal (2023) proposes to reduce the parameters λ and κ derived from the oedometric tests by exactly the settlement-insensitive portion to establish a system stiffness for the Frankfurt Formation as a whole for the respective project site:

$$\lambda_{site} = \left(1 - n_{site}\right)\lambda_{base}\,;\ \kappa_{site} = \left(1 - n_{site}\right)\kappa_{base} \tag{1}$$

where λ_{base}, κ_{base} = base value of compression and swelling index; λ_{site}, κ_{site} = site specific value of compression and swelling index; n_{site} = site specific share of embedded sand and limestone bands.

For the SGZ-Bank the total share of limestone and sand of 17.6% (Ganal, 2023) results in a reduction factor of (1- n_{site}) = 0.824. This approach has already been successfully applied for the back-analysis of the Messeturm (Tafili et al., 2023).

Through the formulation of the AVISA model, the proportion of creep settlements is automatically adjusted in the model itself in the same manner as λ:

$$e_{vis} = \lambda I_v \left(\frac{1}{OCR}\right)^{\frac{1}{I_v}} \tag{2}$$

The critical state slope M_c varies linearly with the depth z [m] below ground surface reflecting a certain spatial variability of the clay which could be attributed for example to the sedimentation process of the material. The data on the overconsolidation ratio *OCR* provided by Franke et al. (1985) and Ganal (2023) are plotted in Figure 5. The *OCR* profile used in the FEA is based on Ganal (2023) and was then slightly adjusted on the basis of preliminary FEA. For the initialization of the horizontal stress by means of the coefficient of earth pressure at rest K_0 (Figure 5), the approach of Mayne and Kulhawy (1982) was used:

$$K_0 = \left(1 - sin(\varphi')\right) \cdot OCR^{sin(\varphi')} \tag{3}$$

5.3 *System permeability*

According to Franzen and Reul (2022), the modelling of the heterogeneous Frankfurt Formation with its embedded limestone and sand layers as a homogeneous aquifer with an equivalent system permeability proves to be an efficient strategy for describing consolidation effects. Applying the approach by Franzen and Reul (2022) the system permeability of the SGZ-Bank site can be estimated evaluating the borehole profiles of the site to be in a range of $k_{sys} = 2.0 \cdot 10^{-10}$ m/s to $k_{sys} = 8.2 \cdot 10^{-10}$ m/s. In the remainder of this paper a homogeneous soil mass with a system permeability for the Frankfurt Formation of $k_{sys} = 2.5 \cdot 10^{-10}$ m/s was applied if not stated otherwise.

Table 2. Material parameters of the AVISA model used for the Frankfurt Formation

Parameter			Base value/ SGZ-Bank
Total unit weight	γ	[kN/m³]	18.5
Buoyant unit weight	γ'	[kN/m³]	8.5
Compression index	λ	[-]	0.108/0.089*
Swelling index	κ	[-]	0.029/0.024*
Poisson's ratio	ν_h	[-]	0.23
Anisotropic coefficient	α	[-]	1.5
Critical state slope	M_c	[-]	0.797+0.0057z
Max. void ratio (p_{ref} =1 kPa)	e_{i0}	[-]	1.44
Loading surface factor	f_{b0}	[-]	1.40
Viscosity index	I_v	[-]	0.030
Viscosity exponent	n_{OCR}	[-]	0.4
Stiffness factor	m_R	[-]	2.5
ISA yield surface radius	R	[-]	$1.8 \cdot 10^{-4}$
ISA hardening parameter	β_0	[-]	0.100
Min. ISA exponent	χ_0	[-]	5
Max. ISA exponent	χ_{max}	[-]	60
Accumulation rate factor	C_α	[-]	0.001

* □ specific value at the SGZ-Bank site
z □ depth below ground surface [m]

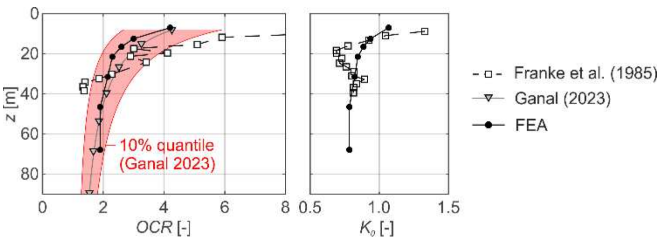

Figure 5. Depth profiles of the overconsolidation ratio OCR and the earth pressure at rest K_0

6 COMPARISON OF NUMERICAL ANALYSIS AND MEASUREMENTS

6.1 *Settlements and angular distortion*

Figure 6 compares the measured settlements and the angular distortion $\alpha_{2/4}$ between MP2 and MP4 with the results of the FEA until the final measurement in 1980. While the FEA slightly overestimates the settlements at all four measuring points during the construction process, in general the time-settlement behaviour is captured reasonably well in the simulation (Figure 6a). When the construction is completed, the average calculated and measured settlements are most identical with $s_{avg,FEA}$ = 19.8 cm and $s_{avg,m}$ = 19.7 cm, respectively. For the last measurement in July 1980 the calculated and measured settlements fall within a range of $s_{2,FEA}$ = 20.0 cm and $s_{4,FEA}$ = 30.0 cm and $s_{2,m}$ = 20.9 cm and $s_{4,m}$ = 30.6 cm, respectively.

For the angular distortion (Figure 6b) both measurements and FEA show the influence of the measures described in section 3. After a sharp decrease of the angular distortion in June 1970 the angular distortion then increases to $\alpha_{2/4,m}$ = 0.23% and $\alpha_{2/4,FEA}$ = 0.25%, respectively, before dropping again in and finally rising to

$\alpha_{2/4,m} = \alpha_{2/4,FEA} = 0.32\%$ in July 1980. The two phases where angular distortion decrease probably correspond to the application of the additional dead weights B1 & B2 (section 3).

The simulated settlement distribution at the end of construction is plotted in Figure 7 for the ground plan of the raft and for a vertical cross-section through the model. The observed tilting of the raft towards the north is well visible in the FEA.

Ganal (2023) compares the results of the initial FEA presented above with an analysis in which none of the measures against the angular distortion of the foundation described in section 3 were modelled. This analysis indicates that while the measures appear to have a significant influence on the angular distortion, the effect on the average settlement of the raft is negligible (Ganal, 2023).

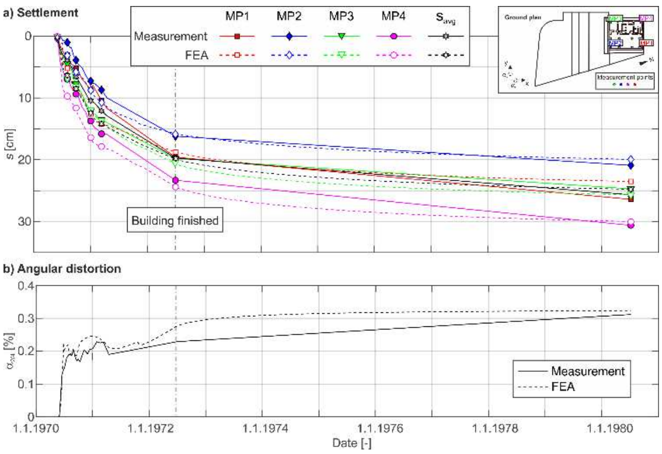

Figure 6. Variation of settlements and angular distortion with time - Comparison of measurements and FEA

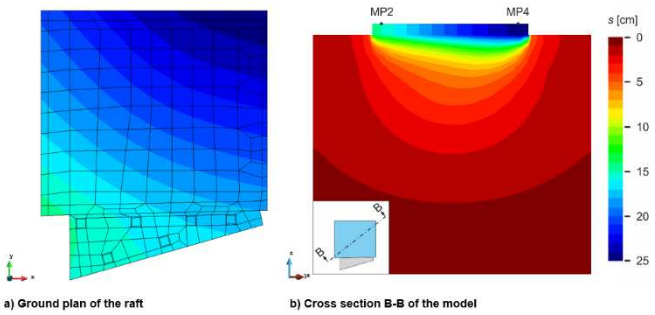

Figure 7. Simulated distribution of settlements at the time of completion of the building (01.07.1972).

6.2 System permeability

Figure 8 shows the simulated excess pore water pressures in a cross section of the model at the date when the largest excess pore water pressure occurred (01.01.1971) and at the time of completion of the SGZ-Bank (30.06.1972). The maximum excess pore pressure can be identified in a depth of 6.5 m below the base of the raft. However, due to the short drainage path the excess pore water pressures at this depth are reduced much faster compared to other parts of the model further away from a drainage surface. Due to the time elapsed between 01.01.1971 and 30.06.1972, the location of the largest excess pore water pressure shifts from a depth of 6.5 m to approximately 12.5 m below the raft and essentially remains there afterwards.

The variation of the system permeability had almost identical effects on all four measurement points investigated, which is why only the development of average settlements is evaluated in the following. Figure 9 compares the variation of average settlements with time for different system permeabilities k_{sys}. For the investigated range of the permeability between $k_{sys} = 1.5{\cdot}10^{-10}$ m/s and $k_{sys} = 4.5{\cdot}10^{-10}$ m/s reasonable agreement is achieved between FEA and measurements.

To investigate the development of excess pore water pressure in the model the average excess pore water pressure Δu_{avg} is plotted in Figure 10 over time. The end of consolidation (*EOP*) is defined at a degree of consolidation $U = 98\%$ with U defined as follows:

$$U = 1 - \frac{\Delta u_{avg}}{\Delta u_{avg,max}} \tag{4}$$

where Δu_{avg} is the average excess pore water pressure over the height of the entire model at the foundation centre and $\Delta u_{avg,max}$ the maximum value of Δu_{avg} occurring over the simulation period.

For a system permeability of $k_{sys} = 4.5{\cdot}10^{-10}$ m/s the *EOP* is already reached on 03.06.1982, whereas for $k_{sys} = 1.5{\cdot}10^{-10}$ m/s the *EOP* is not reached within the observation period until the 01.01.2000 (Figure 10). Approximately from 1992 on, the simulated settlements increase with decreasing permeability for a given point in time. This is due to the rate-dependent formulation of the AVISA model (Tafili, 2020), where for the same stress level, higher settlements are achieved when the material is exposed to lower strain rates.

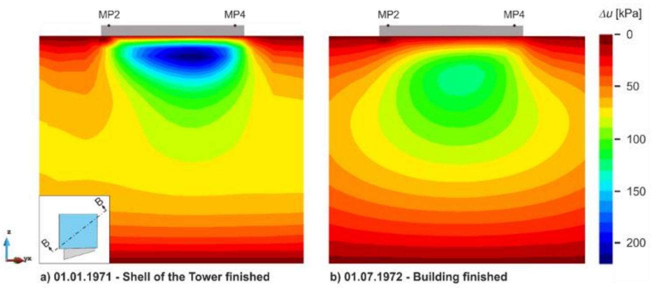

Figure 8. Distribution of the simulated excess pore water at 01.01.1971 and 01.07.1972

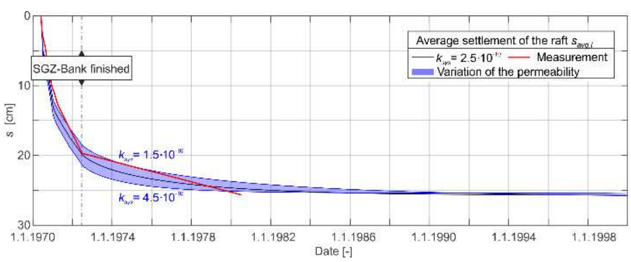

Figure 9. Variation of settlements with time - Influence of the system permeability

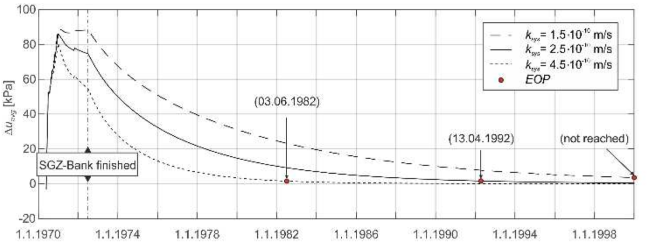

Figure 10. Variation of excess pore pressure with time - Influence of the system permeability

6.3 *Viscosity index*

Figure 11 compares the variation of settlements with time simulated with different viscosity indices varying in a range of $I_v = 0.025$ to $I_v = 0.032$. At the time of completion, almost no influence of the viscosity index on the settlement can be identified. In the first years after completion, different viscosity indices lead to different settlement rates. However, this effect decreases with time which seems plausible considering that the settlements increase linearly over the logarithm of time.

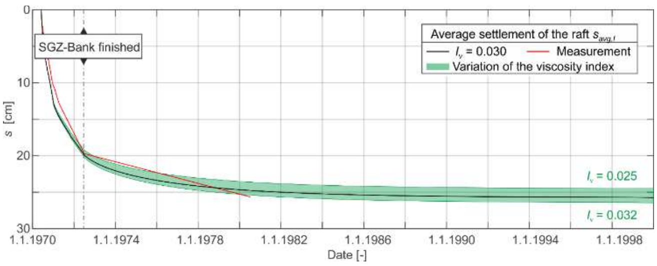

Figure 11. Variation of settlements with time - Influence of the viscosity index

7 CONCLUSIONS

Good agreement between the measured bearing behaviour of the SGZ-Bank in Frankfurt and the results of a 3D coupled pore pressure-displacement FEA was achieved applying the visco-hypoplastic material model AVISA (Tafili and Triantafyllidis, 2020). The approaches proposed by Ganal (2023) for the site-specific adjustment of the soil stiffness by considering the proportion of sand and limestone banks and by Franzen and Reul (2022) for determining a system permeability for the Frankfurt Formation were validated for the SGZ-Bank site.

The FEA indicates that the measures taken during construction of the SGZ-Bank to reduce the differential settlements were mainly successful. The partial demolition of the SGZ-Bank and the reconstruction to the Park Tower will be the subject of further investigations.

8 ACKNOWLEDGEMENTS

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