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High-Frequency Monitoring Results of a Piled Raft Foundation under Wind Loading

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***Abstract*—**Piled raft foundations represent an efficient and reliable technique for transferring high vertical and horizontal loads to the subsoil. Piled raft foundations were successfully implemented for several high-rise buildings worldwide over the last decades. For the structural design of this foundation type the stiffnesses of both the piles and the raft have to be determined for the static (e.g. dead load, live load) and the dynamic load cases (e.g. earthquake). In this context the question often arises, to which proportion wind loads are to be considered as dynamic loads. Usually a piled raft foundation has to be monitored in order to verify the design hypotheses. As an additional benefit, the analysis of this monitoring data may lead to a better understanding of the behaviour of this foundation type for future projects in similar subsoil conditions. In case the measurement frequency is high enough, one may also draw conclusions on the effect of wind loading on the piled raft foundation. For a 41-storey office building in Basel, Switzerland, the preliminary design showed that a piled raft foundation was the best solution to satisfy both design requirements, as well as economic aspects. A high-frequency monitoring of the foundation including pile loads, vertical stresses under the raft, as well as pore water pressures was performed over 5 years. In windy situations the analysis of the measurements shows that the pile load increment due to wind consists of a static and a cyclic load term. As piles and raft react with different stiffnesses under static and dynamic loading, these measurements are useful for the correct definition of stiffnesses of future piled raft foundations. This paper outlines the design strategy and the numerical modelling of the aforementioned piled raft foundation. The measurement results are presented and analysed. Based on the findings, comments and conclusions on the definition of pile and raft stiffnesses for vertical and wind loading are proposed.

***Keywords*—**Dynamic loading, high-frequency monitoring, piled raft foundations, wind loading.

I. Introduction

REALISTIC, economical and safe modelling of piled raft foundations remain a major issue for the geotechnical engineer. Although several empirical, analytical and numerical design methods have been available for decades for this type of foundation (e.g. [1]-[8]), there is not always a good agreement between field measurements and computation results, leaving opportunity for research and development. Most of these design methods usually focus on static load cases. Likewise, German guidelines for piled raft foundations [9] do not deal specifically with dynamic loading. On top of this, measurements of piled raft foundations are mostly available at a low frequency for permanent vertical loading but rarely for dynamic loading such as wind.

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There is already a large body of literature dealing with piled raft foundations (e.g. see bibliography in [9]). While some of the literature is written specifically with regard to piled raft foundations (e.g. [5]-[8]), there is other literature regarding single pile foundations, monopile foundations and pile groups that also describes the effects of cyclic and dynamic loading, as well as design guidelines and recommendations for these conditions. Many of these publications can be attributed to one of the following groups of publications:

(a) Wind and wave loading on wind turbines
(b) Earthquake loading on high-rise buildings
(c) Wind loading of tall chimneys

Due to the interest in alternative forms of energy harvesting in recent decades, an intensive development in the field of wind turbines has continued and has also dealt with questions regarding the foundation of said structures. This research lead to the publication of group (a) (among many others e.g. [10]-[15]).

From group (b), Yamashita [16] monitored the seismic behaviour of a piled raft foundation combined with grid-form deep mixing walls under a 12-storey building in Tokyo before, during and after an earthquake. In [17], a dynamic behavioural study of a building with a piled raft foundation using a time-dependent finite element model is documented. Here they used spring elements to model the soil stiffness. In [18], an assessment of soil-pile-structure interaction influencing seismic response of mid-rise buildings on floating pile foundations, with many similarities to piled raft foundations can be found. Furthermore, the use of multiphase finite element models were evaluated in [19] for calculation of the dynamic impedance of piled raft foundations.

From group (c), many questions concerning the foundation of tall chimneys arise, which are very similar to questions regarding the foundation of high-rise buildings. Often piles or piled raft foundations are used and decisive load cases frequently result from wind loading. An example of this is given in [20], where a soil-structure interaction analysis was carried out for tall reinforced concrete chimneys with piled raft foundations subjected to wind loads.

The following paper outlines the design process, monitoring concept and the resulting monitoring data of Roche Building 1, a 41-storey high-rise office building in Basel, Switzerland. During the design process several questions related to the realistic modelling of the piled raft foundation arose, especially for wind loads. The focus of this paper is the question of whether a wind load is to be taken as a purely dynamic load for the design of the foundation or whether a part of it acts as a static component. It was expected that, among other factors, due to the building inertia and its resonance frequency, a part of the wind load would act as quasi-permanent load whereas the rest would act as a dynamic load. From a geotechnical point of view, the answer to this question influences the subsoil stiffness to be applied for wind load cases. This in turn has an effect on the additional deformations predicted and - of course - on the stress and force distribution changes between raft, piles and subsoil. Having this in mind, the monitoring scheme was designed in such a way, that a regular monitoring was ensured, but high-frequency measurements could be taken as well. This paper attempts to provide additional insight into this topic on the basis of a 5 year high-frequency monitoring campaign of the aforementioned high-rise building foundation.

II. Piled Raft Foundations

A. Development and Behaviour

A piled raft foundation is a hybrid foundation system composed of a raft connected to piles. Both raft and piles are transferring load to the subsoil (Fig. 1). Principles and behaviour of the piled raft foundation are described in the piled raft foundation guideline [9]. The advantage of piled raft foundations over single raft foundations is that the thickness of the raft can be reduced significantly and a decrease of the settlements can be achieved. Compared to a mere pile foundation, the number of piles necessary for the design of a piled raft foundation can be reduced appreciably whereas settlements remain in an admissible range. Piled raft foundations are very interesting from an economical and technical point of view, because they allow the achievement of allowable settlements and design requirements with a lower investment.

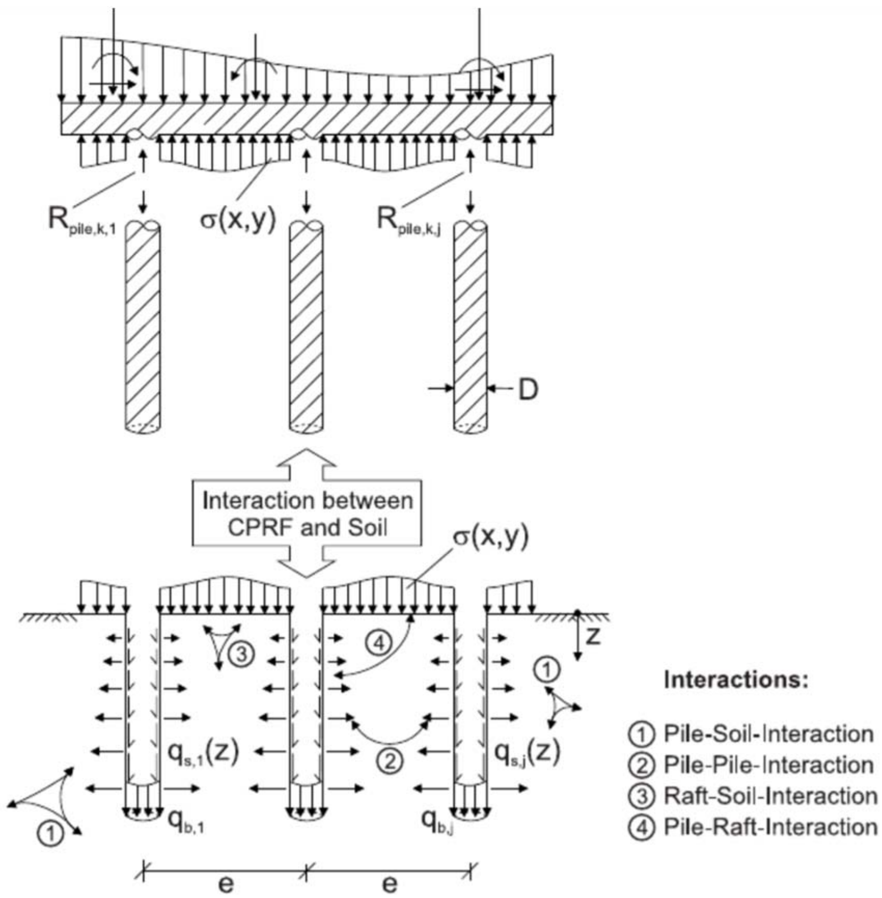


Fig. 1 Behaviour of Piled Raft Foundation [9]

B. Computation

The computation and design of a piled raft foundation is more challenging than the other types of foundations. Usually a sequential procedure is adopted: empirical, analytical or experience based models are used for the preliminary design (e.g. [2], [4]-[7]). For the detailed design, the soil-structure interaction between raft, piles and soil has to be considered while incorporating the load-distribution applied by the building itself. Theoretically, one numerical model simulating the building structure and the foundation including the soil-structure interaction would be preferable. In practice, the numerical codes available are specialised in either geotechnics or superstructure engineering, so that two separate numerical models are usually used: (1) a geotechnical model using a 3D numerical simulation of the structure (raft, piles) and the soil, and (2) a model of the building superstructure itself. The challenge of using two separate models is that the load distribution of the building depends on the stiffness distribution of the foundation, creating a circular reference. This circular reference is usually resolved by an iterative approach.

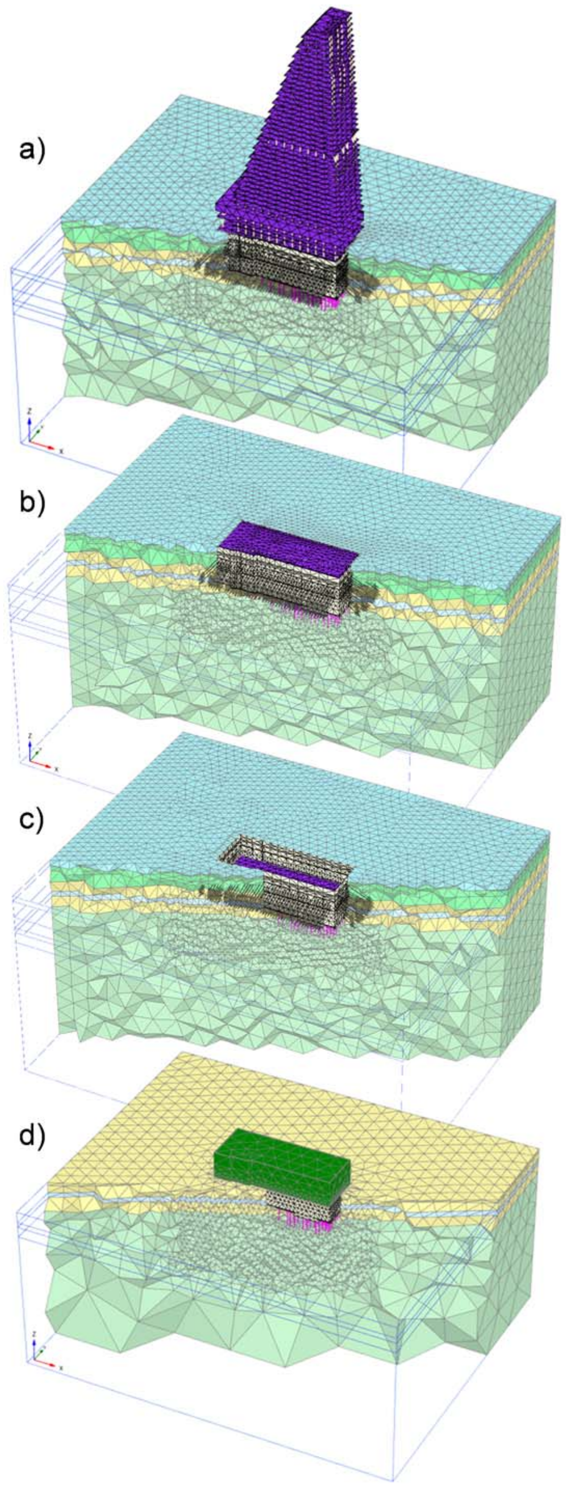


Fig. 2 Modelling of piled raft foundation

Usually the interface between the geotechnical model and the structural model is defined at the bottom-line of the raft, whereas the raft has to be modelled in both systems. As the geotechnical model behaviour is highly non-linear, every single load case has to be computed separately. Therefore a global, single model including both the structural and geotechnical parts (Fig. 2 (a)) is being simplified to a model consisting of raft, piles and soil (Fig. 2 (d)). It would be too time consuming regarding the hundreds of load cases that have to be considered for high-rise buildings, which simulate various loading scenarios for wind and seismic loading. This simplification has shown to be sufficient for the purpose of the foundation design (in the case of the model in Fig. 2, when construction is finished, the simplified model overestimates settlements by ca. 10% whereas the total settlements range at ca. 20 mm). The geotechnical model delivers stiffnesses for every single pile as well as the distribution of moduli of subgrade reaction for the raft, which can be implemented in the structural model. It is recommended to give a lower and an upper bound of values of pile stiffness and moduli to account for uncertainties, variations and simplifications in the model and geotechnical parameters.

The design of piled raft foundations has been performed for decades for vertical loading with a fairly good agreement between computed and measured settlements [21]. Regarding load distribution between piles and raft, however, the agreement between computation and field measurements does not always fulfil expectations. On the one hand, this is the result of difficulty in installing the instrumentation. On the other hand, simplified geotechnical models do not take the pile installation process as well as soil heterogeneity into account.

Earthquake and wind commonly generate the most important horizontal loading on a high-rise building. These types of loads are characterised through a non-constant value and dynamic behaviour. Therefore, modelling has to take the dynamic behaviour of the soil into account. As soils react stiffer under dynamic loading than under static loading, the geotechnical model has to consider the appropriate stiffness for each type and sequence of loading resp. for each modelling phase.

The numerical geotechnical modelling of piled raft foundations is often done using finite element (FE) simulations. These numerical models usually consist of several modelling phases, consecutively simulating the expected loading path. Generalising, the modelling phases may be grouped by:

(1) Modelling the initial stress-strain-field
(2) Realisation of the excavation pit
(3) Drilling of the bored piles, often done during (2)
(4) Construction of the foundation slab
(5) Construction of the building structure including (partial) dismantling of the excavation pit support, activation of service loads
(6) Load cases modelling wind loading, each starting from the last phase of (5)
(7) Load cases modelling earthquake loading, each starting from the last phase of (5)

Often, wind and earthquake loading is modelled using equivalent static loads. When choosing this modelling approach, the model parameters of the subsoil have to be adjusted to represent the dynamic stiffness. All of the cases in phase (7) above are subject to these adjustments. Whether or not it is necessary to adjust for (6) is often open to discussion.

Fig. 3 View out of the excavation pit towards west

III. Case Study

A. Project "Building 1"

In 2015 the company Roche Pharma Ltd. moved to the new office-facility "Building 1" at their headquarters in Basel, Switzerland. With 41 floors and workspace for around 2'000 employees, at 178 m, it is currently the highest office-building in Switzerland. It was designed by the architects Herzog & de Meuron.

For the realisation of the 3 underground floors, an excavation pit with a depth of 19.6 m was necessary. In the elevator areas some deepenings of up to 21.5 m below ground level were required. To limit deformation in the sensitive and densely built surroundings, a retaining wall consisting of a secant bored pile wall with up to 4 anchor layers was necessary.

B. Geological and Hydrogeological Conditions

Fig. 4 contains a geological cross section of the project area [22]. The geological strata can be summarised from a geotechnical point of view beginning at ground level as follows: Topmost is a 2 to 3 m thick layer of fillings (Layer "A" on Fig. 4). Below this lies a thick layer of very dense quaternary sediments ("NTS", mainly gravel and sand) extending down to 16 to 19 m below ground level. These quaternary sediments are over-consolidated because they were once overlaid with an approx. 100 m thick layer of sediments, which were later eroded down to today's level. The fillings and quaternary sediments are only found above the foundation level and are therefore not significant for the foundation design.

The quaternary sediments are underlaid by at least a 100 m thick layer of the Molasse-formation. The Molasse-formation consists of an alternating sequence of sandy Elsässer Molasse ("EM") and clayey Cyrenenmergel (Marl, "CM"). The Elsässer Molasses can be further divided into an upper/ younger ("EM1") and lower/older ("EM2") variant. Selected geotechnical properties of the marl-subsoil can be found in Table I. They were determined on the basis of a laboratory and field testing program including, e.g., triaxial shear tests and crosshole testing [23]. Table II contains bearing capacity properties of bored piles in marl-subsoil (values considered identical for EM and CM, determined on basis of dynamic and static pile testing [24]).

The groundwater table lies about 10 m below ground level and is strongly influenced by the level of the River Rhine, which is approx. 100 m from the project site. Whereas the layers of the Molasse-Formations exhibit very low permeability (1×10^{-6} to 3×10^{-10} m/sec), the quaternary sediments are highly permeable ($1 \ldots 5 \times 10^{-3}$ m/sec).

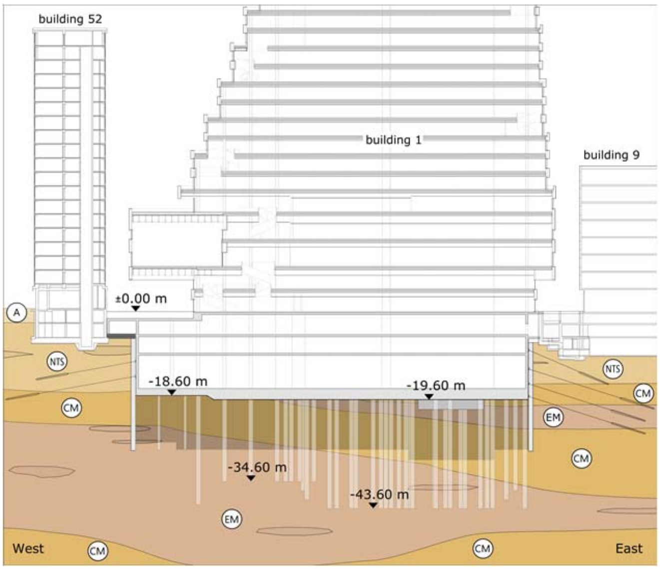


Fig. 4 Geological section (West - East)

TABLE I
GEOTECHNICAL PROPERTIES MARL-SUBSOIL

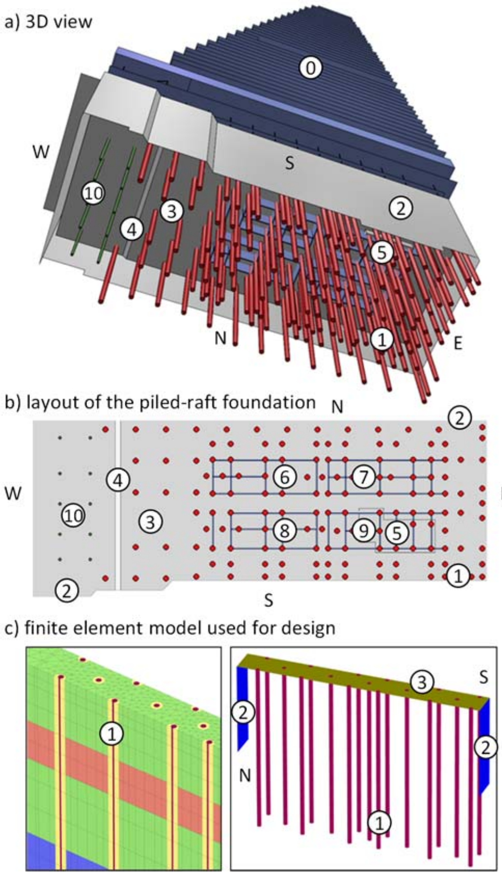
Property	Symbol	Unit	Characteristic value (range) Cyrenenmergel CM	Elsässermolasse EM1 | EM2
Bulk density	γ	kN/m³	22 (20 - 23)	
Wet bulk density	γ'	kN/m³	12 (10 - 13)	
Friction angle	φ'	°	27.5	29
Cohesion	c'	kN/m²	50	75
Permeability	k	m/sec	1×10^{-6} - 3×10^{-10}	
Stiffness oedometric	E_{oed}^{ref}	MN/m²	80	120 | 200
Stiffness un/reloading	E_{ur}^{ref}	MN/m²	200	300 | 500
Dyn. stiffness factor	E_{dyn}/E_{ur}	(-)	3	3 | 2

TABLE II
BEARING CAPACITY PROPERTIES OF BORED PILES IN MARL-SUBSOIL (VALUES CONSIDERED IDENTICAL FOR CM AND EM)

Property	Symbol	Unit	Characteristic value
maximum base resistance	$\sigma_{base,k,pile}$	kN/m²	2500
maximum skin resistance	$\tau_{shaft,k,pile}$	kN/m²	250

C. Foundation Design

Building 1 was built on a piled raft foundation 8[24], [21], [25]. The configuration of the foundation piles shown in Fig. 5 (b) strongly follows the distribution of loads on the top of the foundation slab. These loads are applied by the building's carrying structure, consisting mainly of the four building cores also visible in Fig. 5 (b), and columns. Due to the asymmetric design of the building, the building cores are shifted to the east of the geometric centre. Most of the remaining load is carried by columns to the top of the foundation slab.



⓪ superstructure of building 1
① foundation piles
② bored pile wall
③ foundation slab
④ thickness gradation (1.5 m to 2.5 m)
⑤ lift pit
⑥ walls of north-west building core (projection)
⑦ walls of north-east building core (projection)
⑧ walls of south-west building core (projection)
⑨ walls of south- east building core (projection)
⑩ micropiles

Fig. 5 Foundation layout and numerical modelling

Fig. 5 (a) shows a 3D visualisation of the building including the bored pile wall, foundation slab, and foundation piles from below. The majority of the piles are arranged below the four building cores and the columns. The piles in this area are the longest, reaching up to a maximum of 24 m length. The foundation piles to the west are much shorter (minimum 12 m length) due to the fact that the building loads are much smaller in this area. For the same reason the slab thickness is reduced by 1.0 m from 2.5 m to 1.5 m.

Due to limitations of both the FE software used, and computational power at time of design, a north-south aligned slice-model at the west-end of the two eastern building cores was used for design of the piled raft foundation (Fig. 5 (c)). The decision to use a slice-model was also fuelled by the motivation to reduce the computational cost of one simulation to allow analysis of a larger number of different variants and configurations.

The result of the design process was the pile configuration shown in Fig. 5 (b), consisting of 153 foundation piles with a total length of 2902 m. Due to problems during construction, three piles had to be abandoned and replaced at a slightly different location. Furthermore 10 micropiles were arranged at the west end of the foundation slab to account for groundwater uplifting pressure.

D.Monitoring Design

In accordance with [9] a monitoring concept of the piled raft foundation was developed and installed: As a basis, the vertical displacement of the top surface of the slab was measured at 15 points every two months (marked by triangles △ in Fig. 6). This coarse measurement scheme was chosen due to the high cost of each measurement and the fact that a non-problematic settlement behaviour was expected.

Additionally, several types of measurement sensors were placed below the foundation slab and were monitored at a frequency of 0.2 Hz (1 Hz during manually introduced "high frequency phases"). The selection of these measurement frequencies was made on the basis of various considerations: Firstly, during the design phase, the question arose as to what proportion of the wind load really affects the foundation as a "real dynamic" load. Secondly, the manufacturer of the measuring sensors specified an internal measuring frequency of 1 Hz. Thirdly, there were no technical restrictions regarding storing, transmitting and processing the measured data with the available computer technology.

Installation of the measuring sensors took place in the beginning of March 2012. These 27 sensors also visible in Fig. 6 are:

- 7 pile heads, each instrumented by a load cell for pile forces (P1 to P7, marked by ⊙ in Fig. 6). The pile heads were designed so that the entire load should have been transferred through the load cell, although due to an installation flaw, a part of the load could not be directly measured. This led to a plausible qualitative behaviour of the measured values, which, however, are of too small a magnitude.
- 8 earth pressure cells with thermometry (SD1 to SD8, T01 to T08, marked by ☐ in Fig. 6). The earth pressure cells were placed at a certain distance from the neighbouring piles to avoid the near-field stress field of these piles. Whereas SD2 to SD8 showed plausible measurements, SD1 was unknowingly placed very close to an existing vertical test anchor, leading to an allegedly stiff ground behaviour. Due to their design, the sensors measure total pressures.

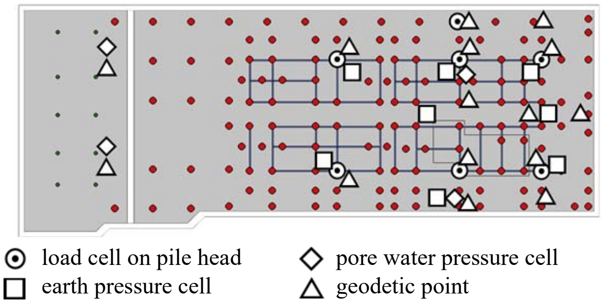


Fig. 6 Monitoring instrumentation

- 4 pore water pressure cells (PW1 to PW4, marked by ◇ in Fig. 6) deliberately not sealed against the water circulating in the cable trenches. Therefore, these pore water pressure cells measured the "global" water pressure at the bottom of the foundation slab, not the pore pressures within the subsoil. For long-term static loading it is expected that the global water pressure and the pore pressures of the subsoil converge. Short-term (e.g. dynamic) loading and/or rapid changes in the groundwater table will on the other hand lead to a temporary difference.

IV. Monitoring Results

A. Results

Recording of the sensor data and the settlements began shortly after installation of the sensors in March 2012. While data recording is still active today (July 2018), only the results up until the end of 2015 are shown in this paper. This range covers the complete time of the construction works and the first months of regular operation of the building. Fig. 7 shows settlement vs. time and Fig. 8 shows the data from the 27 sensors.

The majority of the time data were recorded at a frequency of 0.2 Hz (one measurement every 5 seconds). After the building was completed in 2015, several "high frequency measurement phases" at 1 Hz (one measurement per second) were invoked, each lasting up to 7 days. Hence, in the period from March 2012 to the end of 2015, approximately 24.5 million data sets containing the measured values from the 27 sensors listed in Section III *D* are available.

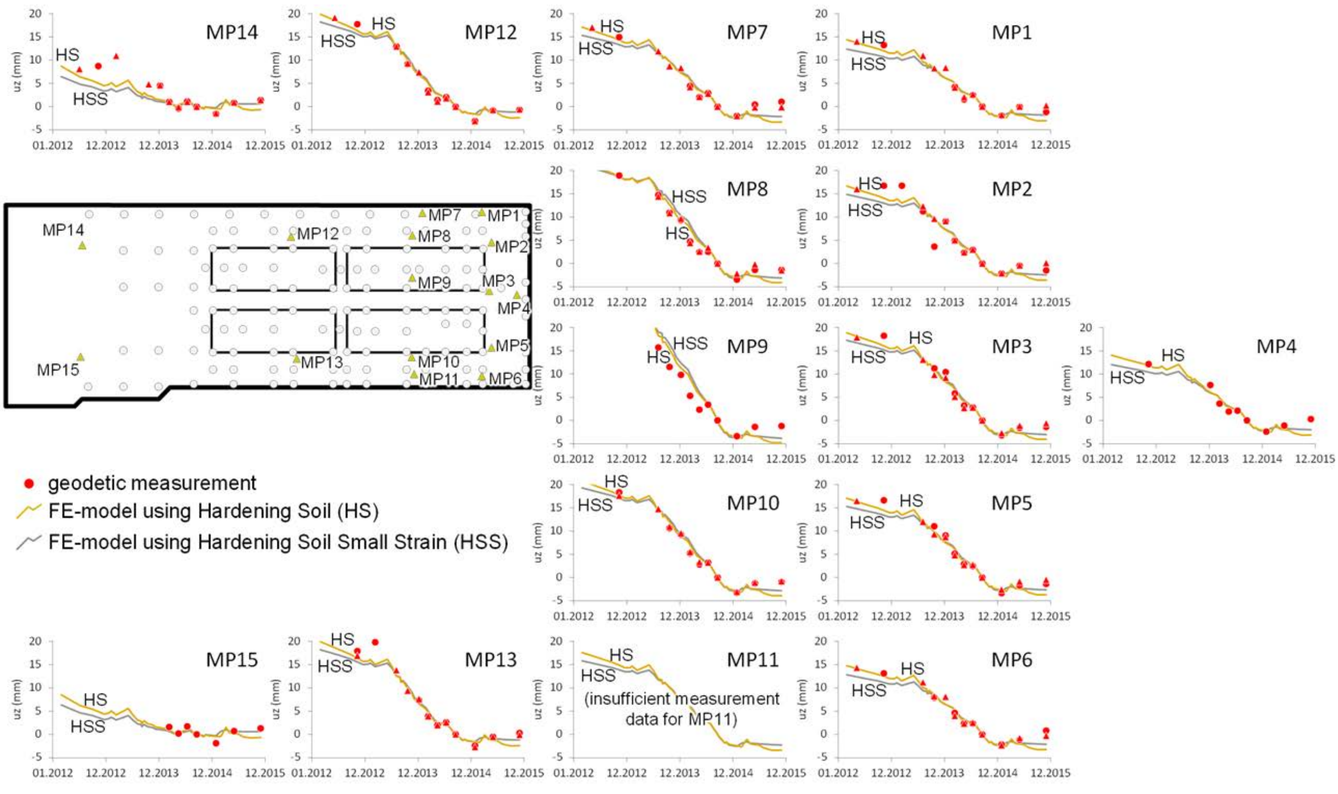


Fig. 7 Measurements and modelled behaviour (settlements vs time)

Due to a planned relocation of the monitoring's field computer, and also due to temporary technical malfunctions, some parts of the data are missing or had to be excluded from later analysis. This explains the gaps visible in the graphs of the monitoring data shown in Fig. 8.

B. Data Analysis: General Behaviour

A very general, but very important statement, which can be derived from Figs. 7 and 8, is that the foundation of Building 1 behaves as expected. This statement can be illustrated by the following observations:

- The settlements in Fig. 7 are increasing gradually to their design value, spatially distributed in the form of a very flat and horizontally aligned subsidence cavity.

- A general rise in pile head forces and earth pressures in accordance with progress of the construction of Building 1 can be seen up until October 2015 (completion of structural work and facade).
- After October 2015, the pile head forces exhibit more or less constant values with only minor noise.
- Analogous to the piles, the measured earth pressure corresponds well to the progress of the construction of Building 1 and show reasonable magnitudes.
- The water pressures recorded reflect the groundwater level. A comparison not shown here with the measured values of a pore water pressure cell outside the excavation pit shows a very good agreement.
- The jump in the measured values of the pore water pressures in February 2013 can be attributed to the deactivation of the dewatering measures.
- As expected, the water pressure on the raft has a direct influence on the earth pressures and the pile head forces. For example an increase in the water pressure leads to a reduction in both the earth pressures and the pile head forces.
- Temperature readings show a very prominent peak over the first few months, which can be attributed to the maturing heat of the foundation slab, which was constructed in phases during that time. In the years following 2012, the temperature readings follow the annual course, which were also observed in the thermal sensors in the area outside the excavation pit.

C. Data Analysis: Wind Loads during Windstorm Event

Between 19.11.2015 and 21.11.2015 a strong wind event took place, during which the measurement frequency was increased from 0.2 Hz to 1.0 Hz. Wind speed and wind direction over time is shown in Fig. 9 and also covers the three subsequent days with lower wind speeds. The strong wind event peaked on 20.10.2015 between 10:00 am and 20:00 pm (UTC±0), when the ground wind speed at the peak was approx. 61.2 km/h. The wind direction showed some variation and was mainly south, southwest and west. In certain time windows the wind blew from the northwest.

The relative changes in the measured physical values of the pile head forces and total earth pressures are small in comparison to their absolute value. After careful consideration and consultation with the manufacturer of the sensors, the measured changes are close to the measurement resolution but still can be considered significant. The following observation can be made:

- Fig. 10 shows the development of the pile forces over this 5-day period. The forces of P02, P03 and P05 are slightly decreasing in the first two days and then rising. P06 increases over the whole five days and P07 decreases during all five days. During the five-day recording, P04 showed malfunction patterns and was therefore excluded from the analysis. It is notable that P01 shows a much smaller magnitude in the changes of the forces. This is attributed to the fact that P01 is the only pile force sensor that is not located under a building core. Since the forces from the wind loads are primarily transferred by the building cores, the forces on P01 have to be transferred via the raft. To carry the wind loads acting as a bending moment in the intersection between building cores and raft, P01 acts accordingly with a longer lever.
- As can be seen in Figs. 11 and 12, total earth pressures and pore water pressures display a decreasing trend during the first two days. They then increase for the following three days. Due to the fact that the earth pressure cells measure total pressure including water pressure, they inherit variation in groundwater level, which is not directly related to the wind event.
- It can also be seen that the pile forces, as well as the earth pressures, have their greatest variation during hours of high wind speed. In addition to the increase and decrease in pile forces and earth pressures, there is also an offset to the trend.
- As mentioned in the section “Monitoring Design” above, SD01 reacts much stiffer and behaves more like a pile due to its probable proximity to a test anchor.
- It is noticeable in Fig. 10 that the piles located under the northern cores (P02 and P03) at 12:00 (UTC±0) on 20.11.215 tend to undergo a temporary positive load increment and at the same time the piles under the southern cores (P05, P06 and P07) undergo a negative load increment. Fig. 9 shows that at this time, winds are coming from the south, which is consistent with the observed pile load variation. This load pattern is interpreted in such a way that the wind load generates a fixed end moment which is then taken up by the piles.

In order to analyse the behaviour of the pile forces under wind conditions, the trend was removed from the pile forces (P04 had to be excluded from analysis due to malfunction of the sensor at this time). We assume that the pile head force can be deconstructed to a sum of different components. If the current trend value is regarded as a baseline (or first component), a quasi-static component and a purely dynamic component are to be summed up. Over time, this purely dynamic component varies between a positive and a negative limit (dyn+ and dyn-).

Fig. 13 illustrates the dynamic and quasi-static part using a section of P02's data set. The quasi-static part is calculated using the mean value of all pile force measurements during five minutes, minus the trend of the pile forces. The dynamic portion is calculated using the maximum or minimum of the pile force measurements during five minutes, minus the mean value during five minutes. At least for the strong wind event of 20.11.2015, the load increase (quasi-static component plus positive dynamic component) is divided into an approx. 40% static and an approx. 60% positive dynamic portion. Analogous evaluations for Roche Building 1 for other wind events showed similar ratios.

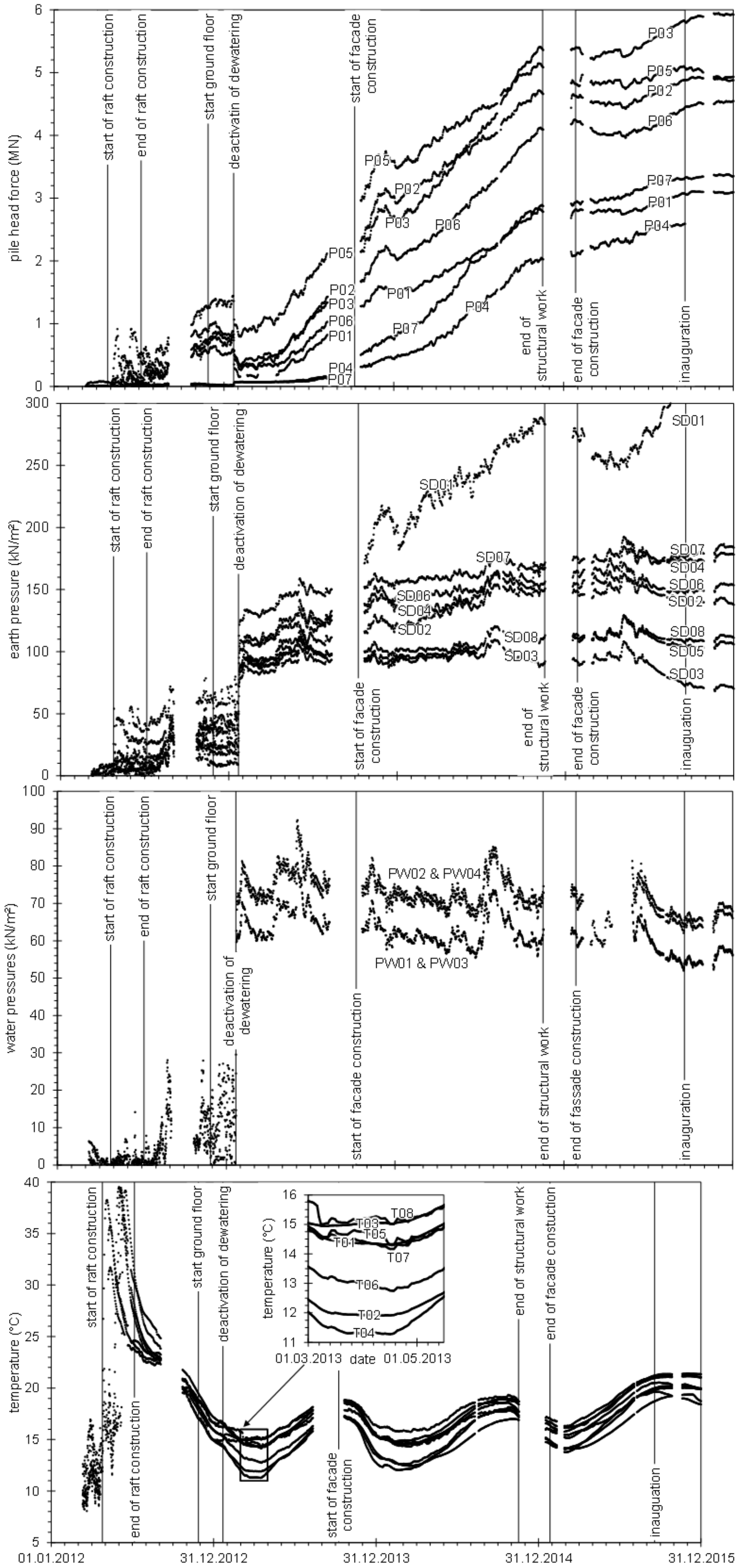


Fig. 8 Measurements (pile head force, total earth pressure, water pressure and temperature vs time)

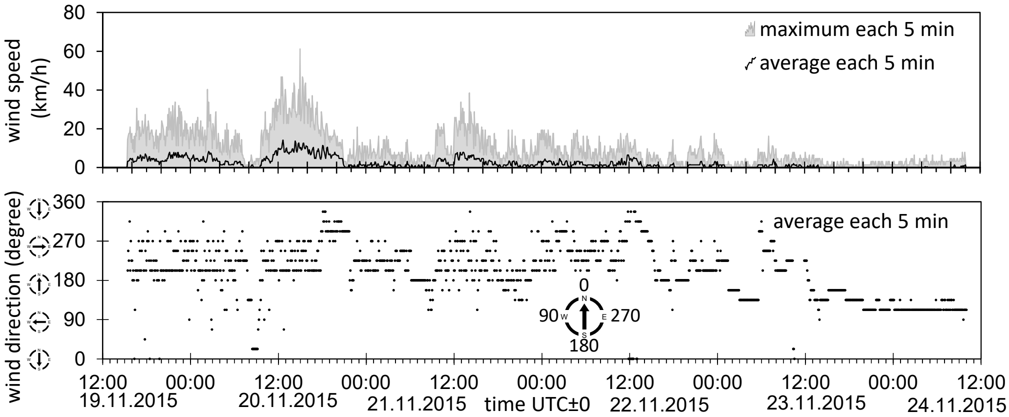


Fig. 9 Wind speed and direction during strong wind event

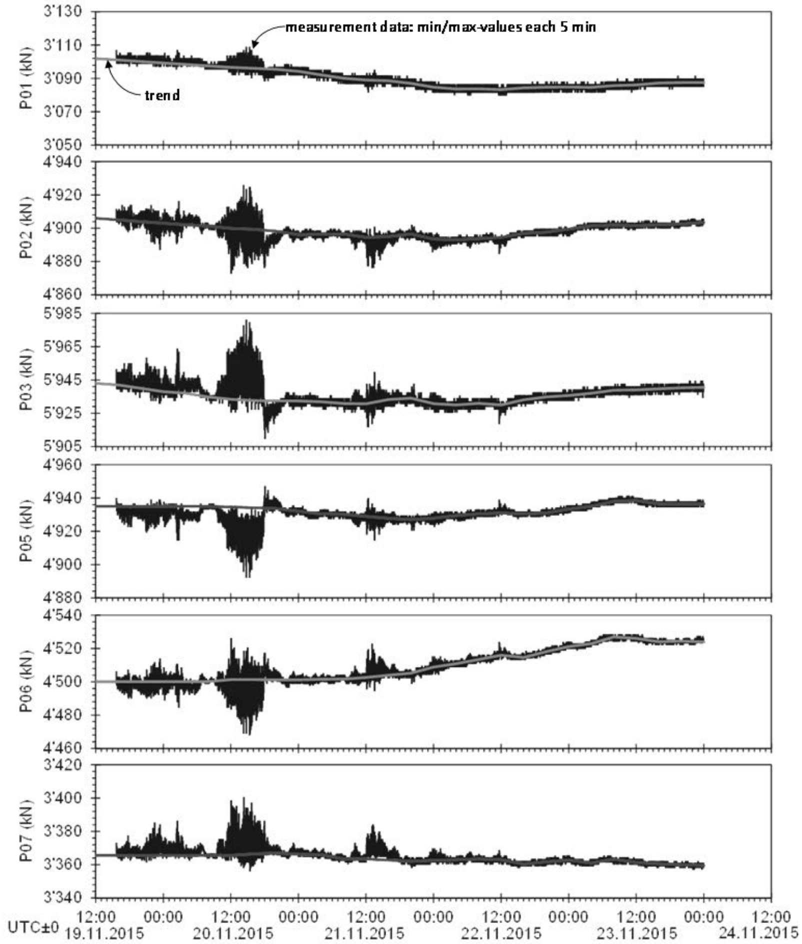


Fig. 10 Variation of pile forces during strong wind event

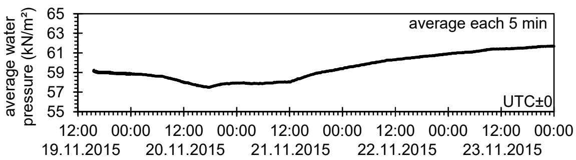


Fig. 11 Variation of water pressure during strong wind event

D.Data Analysis: Estimation of the Natural Frequency

It is expected that the building will oscillate with its natural frequency after a single gust of wind and that this oscillation can also be measured at the foundation-level. From the point of view of the pile forces, this should manifest itself in small additional and reduced loads that vary over time. In the time series of the high wind speed event, a corresponding pattern of measured values was found several times. The period of 25 seconds starting on 20.11.2015, 13:54 o'clock, was chosen for analysis (Fig. 14).

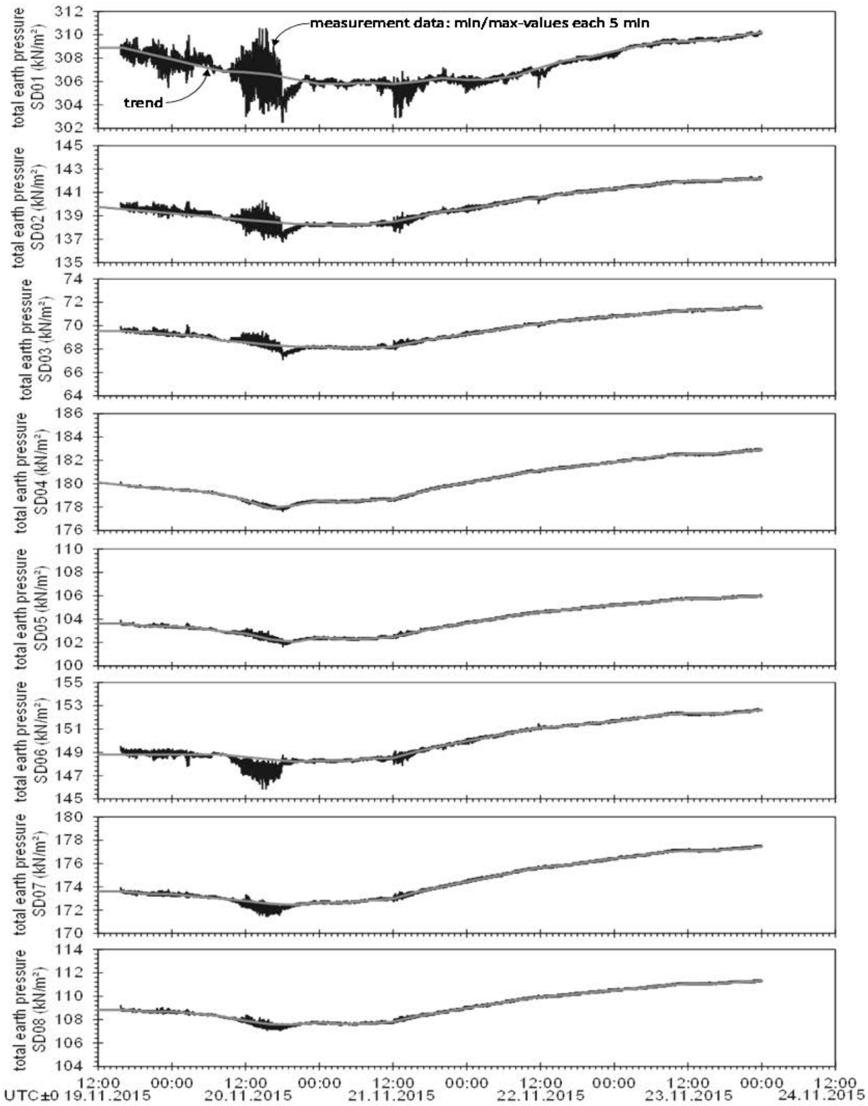


Fig. 12 Variation of total earth pressure during strong wind event

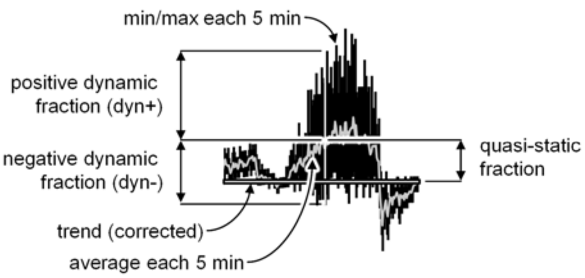


Fig. 13 Decomposition of the pile forces into a static and a dynamic fraction

At the beginning between 13:54:05 and 13:54:10 a change in the pile forces of approx. 9 kN to 14 kN is visible, which is most likely caused by a gust of wind. Afterwards the pile forces oscillate with an amplitude of approx. 3 kN and a frequency of 0.35 Hz.

It can be assumed that the oscillation frequency is the natural frequency of the building in the excited direction. The wind measurements show a wind direction of 180° (south) during this period. Thus, 0.35 Hz seems to be the natural frequency in the north-south direction.

If frequencies in general are to be reconstructed after time-discrete sampling, the data must be sampled at a frequency that is greater than twice the highest frequency occurring in the signal (sampling theorem; Nyquist Shannon theorem or WKS theorem after Whittaker, Kote-lnikow and Shannon). Since the measurement results have a resolution of 1 Hz in the period under consideration, a maximum oscillation frequency of 0.5 Hz can be reconstructed. Therefore, the frequency occurring in the signal is in the traceable range at 0.34 Hz. This frequency was confirmed by the structural engineer, who uses accelerometers to measure a natural frequency of 0.34 Hz (North-South) and 0.58 Hz (East-West).

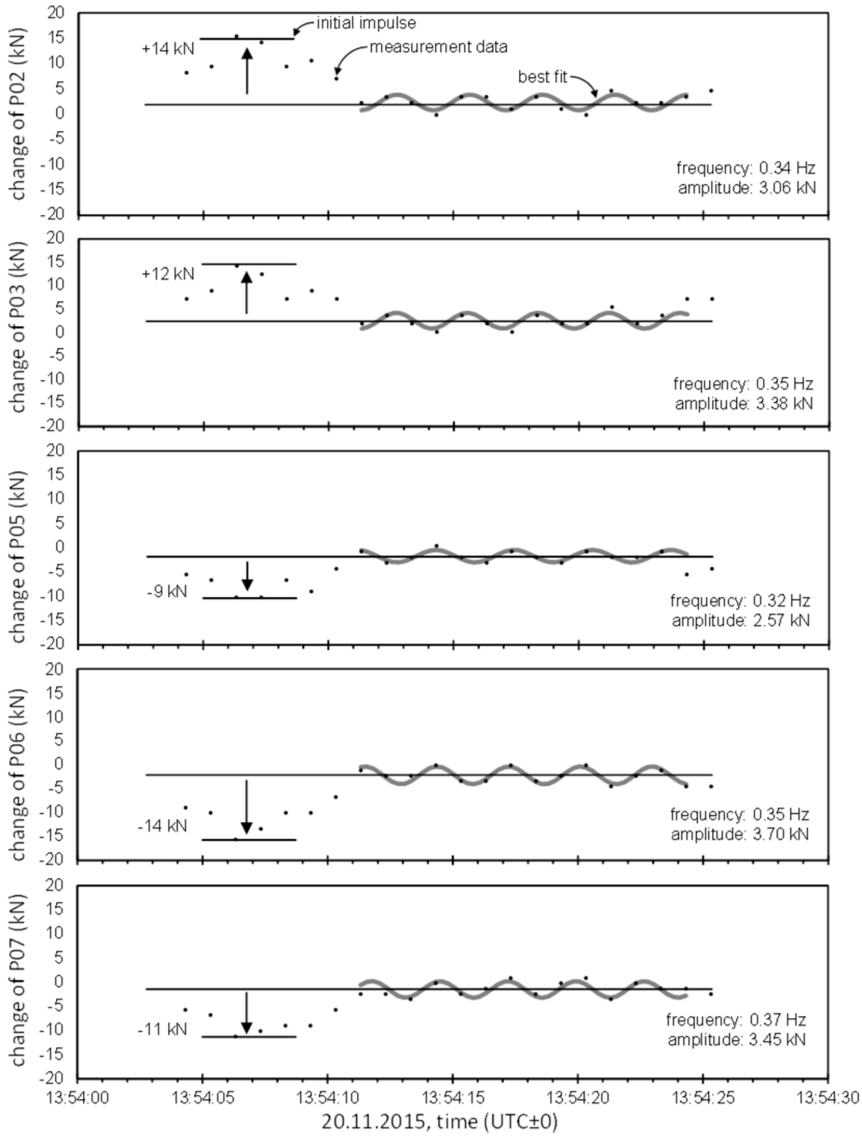


Fig. 14 Estimation of the natural frequency based on the excitation due to a gust of wind

E. Analysis: Back Calculation

A back calculation of the soil parameters of the molasse under Building 1 was carried out due to the fact that other high-rise buildings are planned on the same site. Aside from the fact that the measured settlements fit the predicted settlements as visible in Fig. 7, one of the main goals of the back-calculation was to verify and calibrate the "Hardening Soil Small Strain" material model, as the older "Hardening Soil" model was used during the design of Building 1.

The model shown in Fig. 2 (d) was used to perform the back-calculation. It should be explicitly pointed out that a 3D model of the entire foundation and the subsoil was the basis for this back-calculation.

Fig. 7 shows settlement over time for all 15 measurement points, both measured (red dots) and simulated (final and calibrated parameter set, solid lines). Whereas most features of the measured data are comprehensible and could be replicated by the numerical model, the true source of some features are unclear and are also not represented in the simulation results (e.g. heave measurements during the last measurement campaign). From the authors' point of view, however, the achieved agreement between model and real measured values is very good, allowing both material models to be used for future projects.

V. Recommendations

The authors would like to give the following recommendations regarding the design of piled raft foundations:

- 3D numerical models: With the numerical tools currently available, it is still necessary to create two separate models (1) for building superstructure and (2) geotechnics. For the geotechnical model, a full 3D model of the foundation and the surrounding subsoil should be used for the simulation.
- Automatic iteration: Although this technology has not

been explicitly discussed in this paper, an automatic iteration between the numerical model of the superstructure and the geotechnical model is strongly recommended to mitigate the otherwise labour-intensive and error-prone manual iteration process.

- Subsoil stiffness for wind loading: Despite the growing performance of computer technology, in the near future the authors expect dynamic loads – like wind loads – will still be taken into account by means of (static) equivalent loads. Therefore, the subsoil stiffness has to be adapted in the numerical model to correctly represent the (usually higher) dynamic stiffness.

In the numerical model, a wind load can be applied in two ways: (1) using two phases, the static load component being applied in the first phase using the static stiffness parameters and the dynamic load component in a second phase using the dynamic stiffness parameters. Alternatively, (2) the wind load can be applied as a sum of static and dynamic load components in one phase, in which case the soil stiffness must be adjusted proportionally to the ratio between static and dynamic load component.

In view of the results of the investigations presented here, in similar conditions (type of wind, building configuration, etc.) we recommend dividing the wind loads into quasi-static and dynamic parts, depending on the permeability of the soil. For sufficiently permeable subsoil which allows a dissipation of pore water pressure during the wind loading we recommend an approx. 40% static and an approx. 60% dynamic component. For impermeable subsoil we recommend considering the load as being 100% dynamic.

The authors would also like to give the following recommendations regarding monitoring of piled raft foundations:

- Undertake high-frequency measurements whenever possible: Usually, the monitoring sensors installed are able to give readings at high frequency (often 1 Hz). When designing a piled raft foundation monitoring system, it should be designed to support these maximum data rates by default. With the computer technology available this should be possible without any additional cost. High-frequency data allow much better insights into short-term influences and/or malfunctions that may be misinterpreted at a coarser measurement frequency.
- Automatic settlement measurements: When possible, settlements should also be measured automatically to allow a high measurement frequency. Settlements are an important aspect of the data interpretation for a piled raft foundation and help to understand the underlying processes significantly.
- Measurements of the groundwater level: Often not considered in automatic monitoring, the groundwater level significantly influences the load-bearing behaviour of a piled raft foundation and should be measured in conjunction.
- Monitoring period: Monitoring should be started as soon as possible after the installation of the sensors and should cover the complete construction process and the first years of operation.

VI. Conclusion

Although more and more piled raft foundations are being constructed, very little data are available regarding their response to wind loading. With this paper, the authors hope to contribute to the understanding of this aspect of the behaviour of piled raft foundations. For a high-rise building in Switzerland, monitoring data of the first years of the building's existence, including its construction phase, have been presented and interpreted regarding selected aspects. Particular attention was paid to the data of monitoring phases during periods of high wind speed where the monitoring frequency was increased to 1 Hz.

Based on the results of the data and its analysis, and also based on the experience of the authors, several recommendations regarding design and monitoring of piled raft foundations have been compiled.

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