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CYCLIC PRELOADING OF PILES TO MINIMIZE (DIFFERENTIAL) SETTLEMENTS OF HIGH-RISE BUILDINGS

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ABSTRACT

Preloading and cyclic unloading and reloading of piles is an innovative method to reduce total and differential settlements of high-rise buildings and/or statically sensitive structures. It is performed by using the piled raft or capping structure as a counterweight. This is demonstrated for the 202 m-high Millennium Tower in Vienna and illustrates the different load-settlement behaviour of the individual piles before preloading. The maximum total settlement of the Tower could be reduced to 38 mm. Moreover, complicated re-adjusting devices between the Tower, the annex and the watertight basement joints below groundwater level could be avoided.

KEY WORDS

- *Pile preloading*
- *Settlement minimization*
- *Pile-raft foundations*
- *Bearing capacity of piles*
- *High-rise buildings*

INTRODUCTION

From numerous full-scale tests it is well known that the bearing behaviour of individual piles on a site usually differs more or less widely. This may cause stress constraints within the structure, stress-redistribution with local overloading, and differential settlements. To avoid this and to also reduce absolute settlement, a preloading and load cycling technique was developed, which comprises all structural piles of a building without hindering the construction work. The piles are preloaded as single elements or in groups by imposing a load which exceeds the design load by at least 20 %. Flat jacks have to be placed on the upper end of the piles, and equipment for post-grouting should be installed if the jacks remain within the structure. This provides the final bond between the piles and raft after finishing the entire building.

The basic idea is not only to minimize the total settlement of the pile foundation or the piled raft foundation, but also to achieve a uniform load-movement behaviour of all the piles. Experience has shown that in most cases this requires hysteresis loops with at least two to three cycles of (re)loading - unloading. The aim is to have rather similar gradients along the statically relevant section of the final load-settlement curves under service conditions.

PROJECT

The Millennium Tower in Vienna comprises a 202 m-high main building (Fig. 1) surrounded by a structure of eight floors, three of them below groundwater level. The core of this architectonically and structurally connected assembly rests on a piled raft foundation (area = 1600 m^2), and the adjacent parts are founded on a conventional raft. The excavation of the 15 m deep construction pit within the urban area required a 25 to 30 m deep slurry trench wall, which was tied back with prestressed anchors. It did not reach into the clayey aquitard, but was designed as an imperfect cut-off wall embedded in silty sand and sandy silt. Therefore, 12 double-level wells were needed within the cut-off to lower the groundwater in the quaternary and tertiary sediments.

SLOVAK JOURNAL OF CIVIL ENGINEERING

2005/3 PAGES 1 — 12

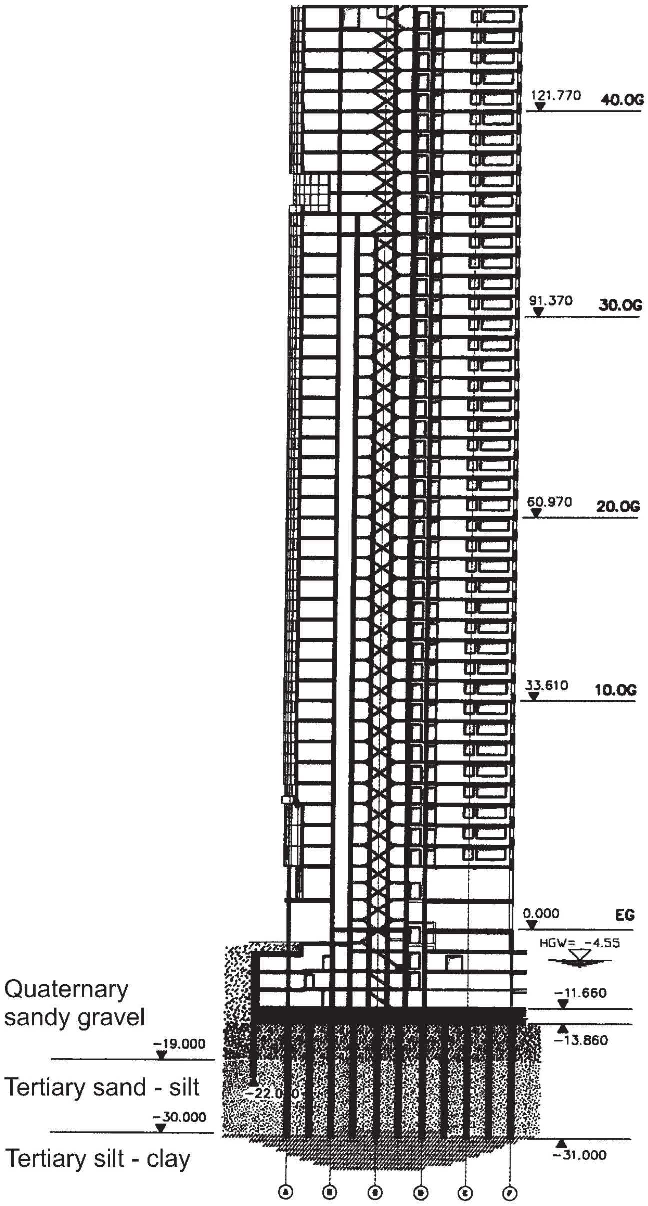

Fig. 1 *Partial cross section trough the main structure of the 202 m high Millennium Tower in Vienna. Adjacent structures are not shown.*

The original design of the high-rise building and its surrounding structure involved fairly complicated relevelling and adjustment equipment to allow for the compensation of possible differential settlements between the structural units. This would have required long-term monitoring and continuous adjusting. To avoid this, an alternative was developed: Local soil improvement by deep vibrocompaction and preloading (overloading and load cycling) of all piles. The main purpose of the vibroflotation was to homogenize and densify the near-surface cohesionless subsoil and increase the portion of the load directly transferred from the raft to the ground. Furthermore, the vibroflotation should have improved the subgrade reaction below the raft and anticipated certain earthquake-induced dynamic forces in the ground.

GROUND PROPERTIES

The soil characteristics were determined by field and laboratory tests (soundings, borings). The investigations disclosed that the overall soil properties were similar, but exhibited a relatively wide scatter in their detail. The subsoil consists of young fills (manmade) and fine-grained river deposits underlain by quaternary sandy gravel, which reach 3 to 9 m below the raft (6 – 8 m within the perimeter of the core structure). Figure 2 illustrates the heterogeneity of the quaternary sandy gravels as determined by heavy dynamic probing. Former branches of the Danube River contributed significantly to the heterogeneity of the ground.

The underlying tertiary sediments vary from uniform silty sands to silty clay, whereas silty sand and sandy silt of zero to low plasticity predominate in the upper zone and silt to clay in the underlying strata. The odeometric modulus of these layers varies between E_{s1} = 8 to 43 MN/m^2 for the first loading and about E_{s2} = 40 to 90 MN/m^2 for reloading. Due to the overconsolidation of the tertiary sediments, the E_{s2} values represent the relevant data with regard to the pre- and re-loading procedure for the foundation piles. The angle of internal friction is Φ = 20 to 28°, and in exceptional cases, up to Φ = 32°.

The groundwater level was originally close to the ground surface and corresponded to the water level of the nearby Danube River. Since installation of the cut-off wall, it now lies 6 m below the surface.

LOCAL SOIL IMPROVEMENT; PILE INSTALLATION, PRELOADING, AND LOADING CYCLING

The 202 m high core structure of the Millennium Tower was founded on 151 continuous flight auger piles (d = 0.88m) of 13 to 16 m length. Before piling, the heterogeneous sandy gravel was locally improved by vibroflotation if the penetration resistance of the heavy dynamic probing was less than n_{10} = 20. Vibroflotation

SLOVAK JOURNAL OF CIVIL ENGINEERING

2005/3 PAGES 1 — 12

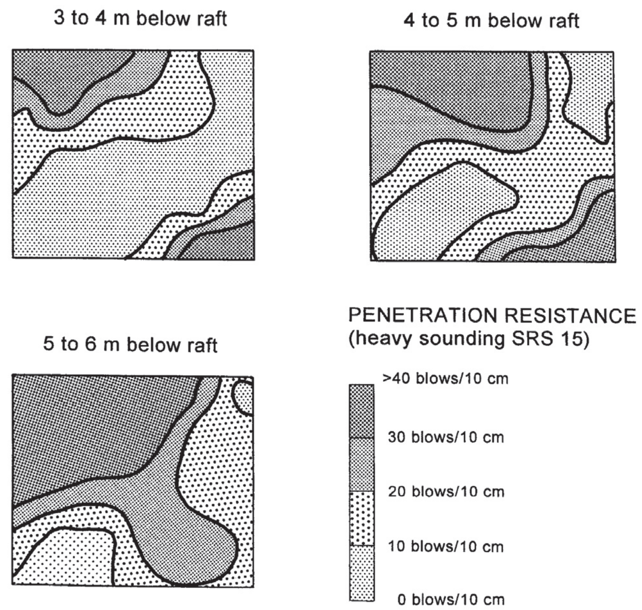

Fig. 2 *Iso-areas of penetration resistance for heavy dynamic probing across the central site of the Millennium Tower (39 x 40.5 m).*

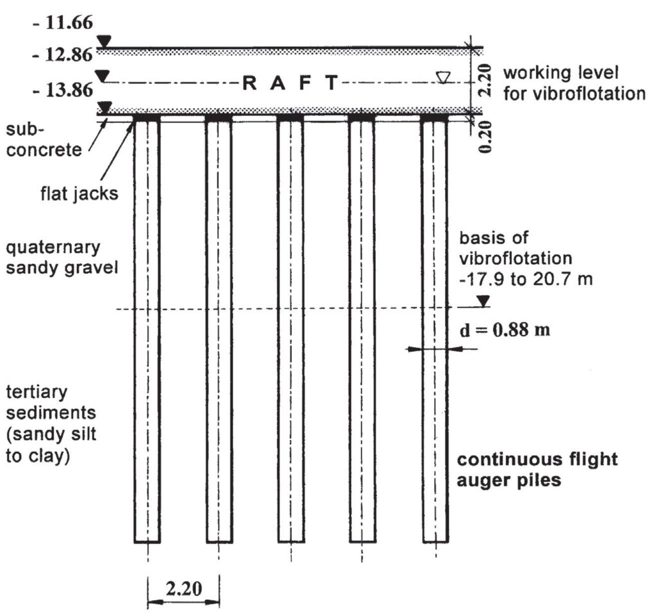

Fig. 3 *Scheme of the piled raft foundation of the Millennium Tower with local vibroflotation to achieve a quasi-composite body, and flat jacks on top of the piles for subsequent preloading and load cycling.*

was performed on a working level 1.2 m above the pile heads (Fig. 3). The mean spacing of the vibroflotation points was 2.5 x 2.5 m. After homogenizing the near-surface ground, the piles were installed from a working level, which was protected by 0.2 m of sub-concrete (reinforced with wire mesh). Experience from several sites has disclosed that such surface protection improves the load-bearing behaviour of pile groups: On several sites an increase of 10 to 20 % in bearing capacity has been measured under comparative conditions.

The 2.2 m thick raft of the high-rise core structure was constructed without joints. The steel reinforcement was 185 kg/m^3 and the concrete quality C30/40. The jointless raft required a casting of 3520 m^3 concrete without interruption over 15 hours.

Preloading of the piles started about three months after their installation, when the second sub-floor of the building was already under construction. Until then no relevant load transfer into the piles had occurred: the loads were running from the raft directly into the

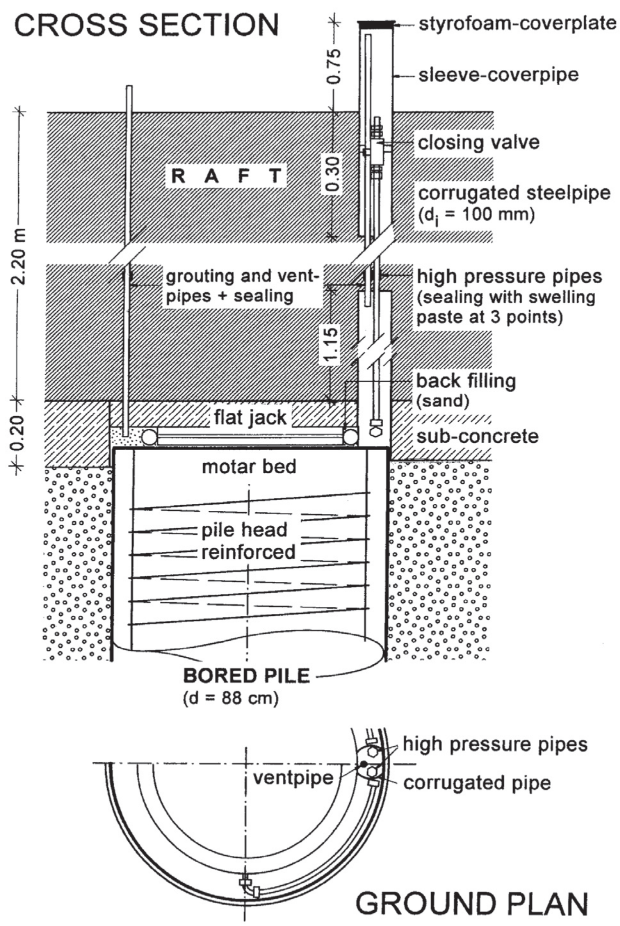

Fig. 4 *Detail of Fig. 3: Pile preloading and load cycling system.*

SLOVAK JOURNAL
OF
CIVIL ENGINEERING

2005/3 PAGES 1 — 12

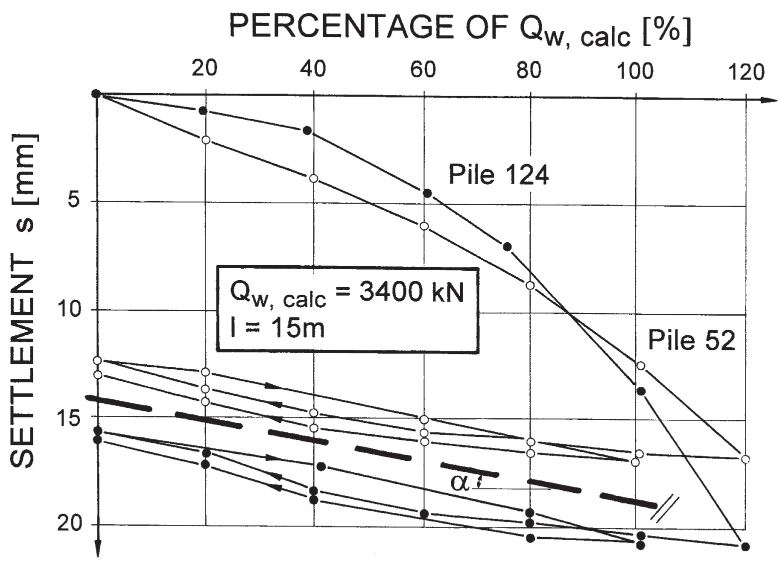

Fig. 5 *Load settlement curves for two relevant piles of the Vienna Millennium Tower, illustrating the locally different pile behaviour. Preloading and load cycling procedure until uniform gradients of the final reloading curves are achieved.*

$Q_{w,calc}$ = calculated working load (under service conditions)

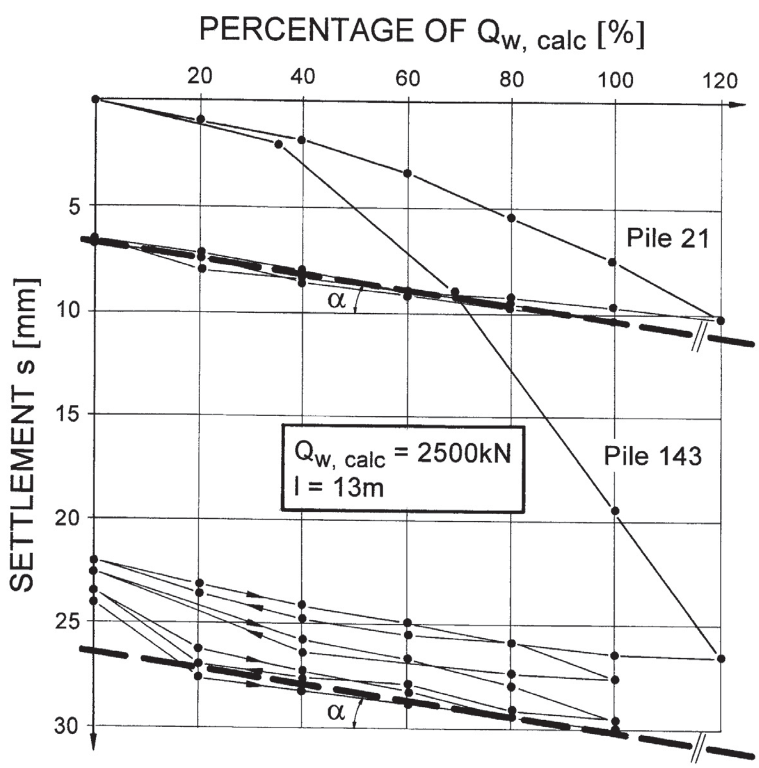

Fig. 6 *Similar to 5, but two other piles with different lengths and $Q_{w,calc}$*

SLOVAK JOURNAL OF CIVIL ENGINEERING

2005/3 PAGES 1 — 12

ground. This could be achieved by an open space between the pile head and raft into which a flat jack was placed (Fig. 4).

Preloading of the piles required flat jacks of a 600 ton capacity and a 50 mm vertical lift range; their diameter corresponded to the pile diameter minus the concrete cover and reinforcement, hence 700 mm. Each of the 151 piles was preloaded, either as a single element or in groups with a maximum of 10 piles. The space between the piles, which underwent simultaneous loading, was 7.5 m to 10 m in order to prevent mutual influencing. This procedure did not hinder any on-going construction work, and it did not require any additional counterweight besides the dead load of the raft. In total, it was performed over 15 working days, i.e., within three weeks. About three weeks after preloading, the hydraulic oil in most of the flat jacks was replaced by a high pressure injection of cement. This provided a stiff connection between the raft and piles. Eleven pile heads were left as measuring piles to collect as much data as possible under the total load of the building.

Preloading of the piles involved a first loading of up to 1.2 $Q_{W,calc}$, where $Q_{W,calc}$ is the calculated working load under the service conditions of the building (design load). This temporary overloading was followed by unloading - reloading cycles until the gradients of the reloading curves were practically equal. Figures 5 and 6 show this for two piles, each with a rather different load-settlement behaviour during the first loading. A conventional foundation technique without any preloading would have caused strong constraints within the piled raft under building loads, because the pile characteristics differed so much. This could now be eliminated by means of preloading and load cycling.

The preloading of each pile disclosed very clearly that there is a relatively great difference in the load-settlement behaviour of individual piles on the same site, despite rather similar ground conditions, the same type and dimension of piles, and the same rig and crew. There were no identical piles, and the scattering of their deformation behaviour increased with the load, as illustrated in Figure 7.

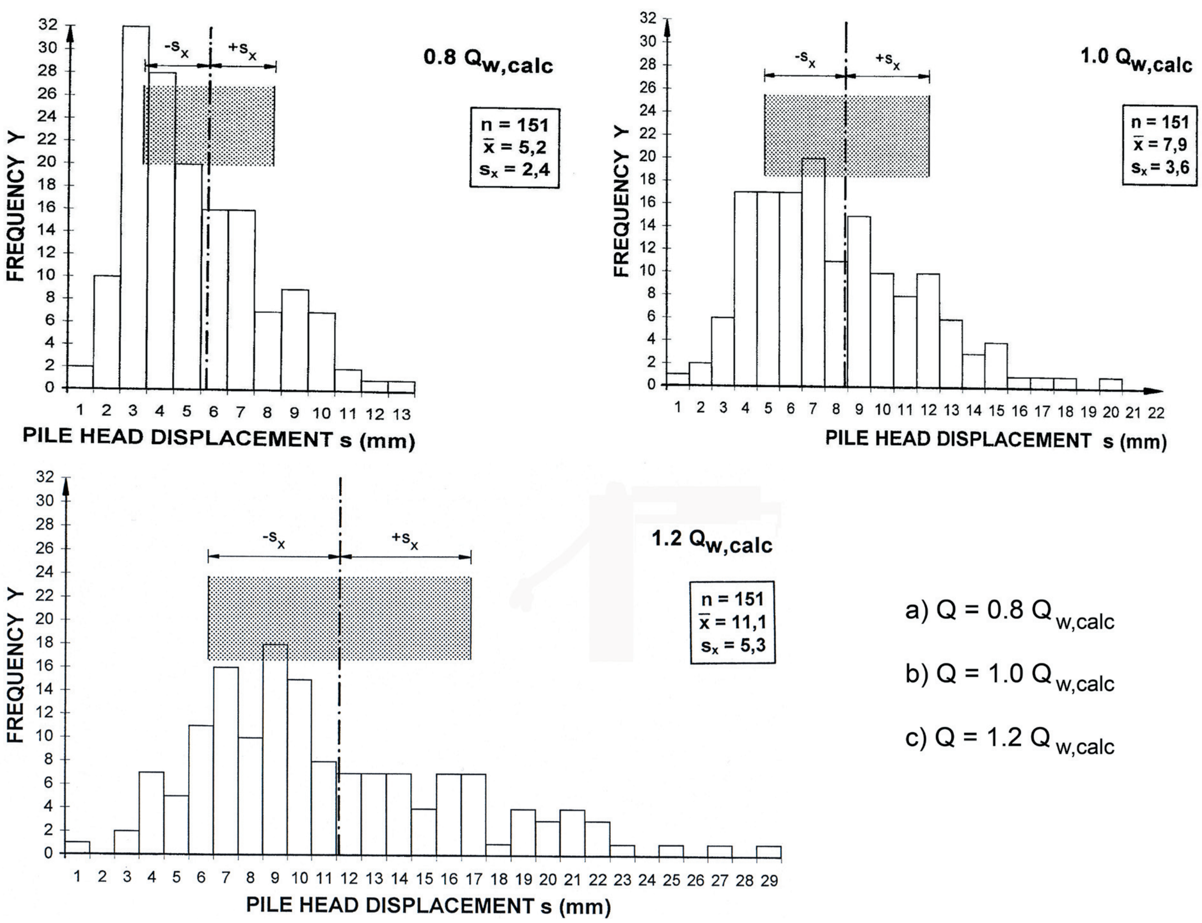

Fig. 7 *Histograms for pile head settlement of 151 piles of the Tower under first loading for different load steps.* $Q_{w,calc}$ *see Fig. 5.*

SLOVAK JOURNAL OF CIVIL ENGINEERING

2005/3 PAGES 1 — 12

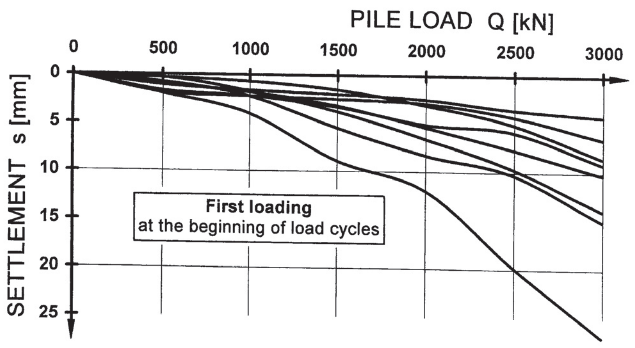

Fig. 8 *Load-settlement curves of eight adjacent piles under first loading (= preloading).*

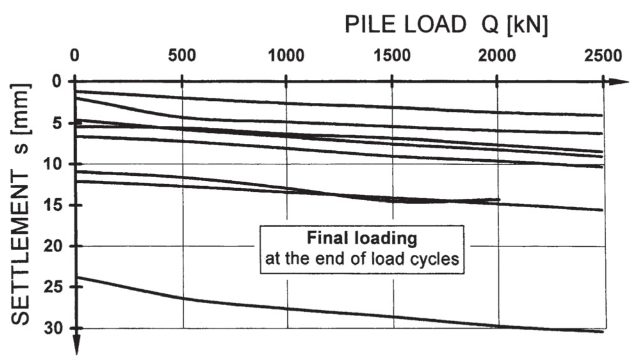

Fig. 9 *Load-settlement curves of the same piles as in Fig. 8, but at the end of the load cycling.*

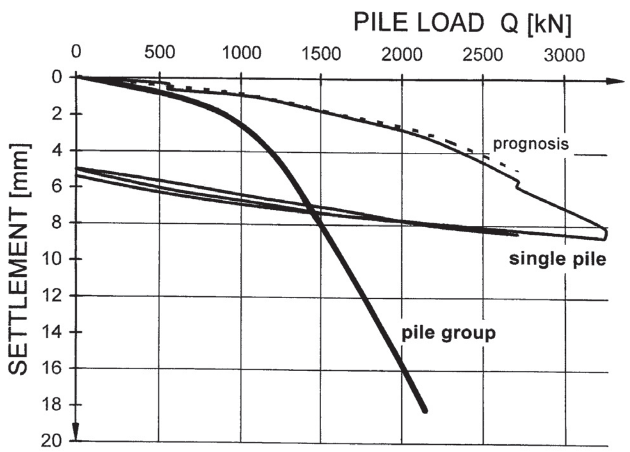

Fig. 10 *Load-settlement curves of the central measuring pile: Behaviour as a single element during preloading (prognosis and measurement) and as a group element under structural load (measurement).*

Figure 8 shows the load-settlement curves of eight neighbouring piles under first loading, and Figure 9 gives the corresponding curves after preloading and load cycling. The final load-settlement curves which are relevant to pile behaviour under building loads exhibit a fairly similar gradient within the statically relevant load ranges.

COMPARISON OF A SINGLE PILE AND PILE GROUPS; PILE-RAFT INTERACTION

One of the central piles was adapted as a measuring pile with strain gauges, pressure cells, and a multiple extensometer. The data gained from the vibrating wire technology were recorded in 1'-intervals during the preloading and load cycling procedures and in 1^h-intervals during the entire construction period of the high-rise building. Despite the rough site conditions, the entire measuring equipment still works and no cell has yet failed.

The measurements disclosed that a cyclic precompression of the soil beneath a pile base increases its bearing capacity significantly. Figure 10 illustrates the great difference between the behaviour of the central measuring pile as a single element (during preloading) on the one hand and as a group element (under building loads) on the other hand.

The distribution of the skin friction (i.e., shear resistance) along the pile is given in Figures 11 and 12. The measurements along three sections show different results for the pile acting as a single element (during the preloading and load cycling) and for the same pile under building conditions (group effect). Furthermore, they illustrate the strong effect of the vibrofloatation, which resulted in the creation of a composite body in the upper zone of the ground. The quasi-monolithic behaviour of the top 5.5 m under building conditions (i.e. pile group behaviour) involves practically no relevant differential movement between the piles and soil, hence hardly any skin friction. Simultaneously, load is transferred more directly from the raft into the ground, and the deeper zones of the piles are more intensively mobilized for load transfer than in the case of the single pile state.

Figure 12 shows the decrease in the normal force, Q, in the pile with depth, which was measured for a pile load of 0.5 $Q_{W,calc}$ and 0.8 $Q_{W,calc}$. The curves clearly disclose that the load transfer from the pile into the ground (by skin friction) occurs at a greater depth under

SLOVAK JOURNAL OF CIVIL ENGINEERING

2005/3 PAGES 1 — 12

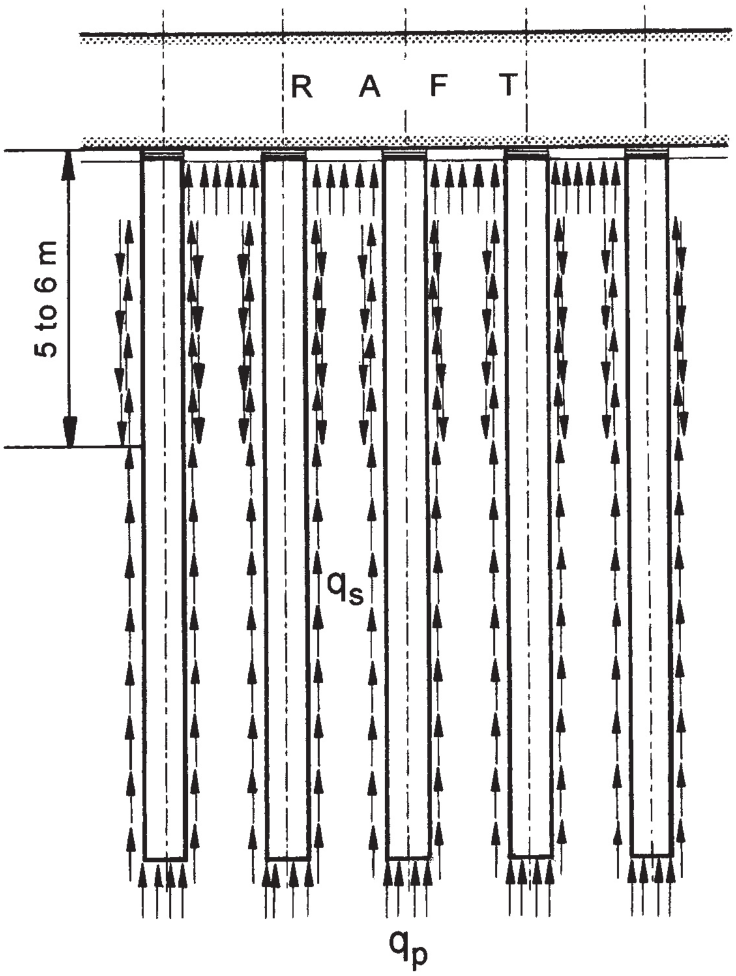

Fig. 11 *Load transfer from the piled raft foundation into the ground. Schematic.*

a group condition. The load transfer from the raft to the ground occurs mainly in the topmost 5 to 6 m, where the negative skin friction (from the raft) and positive skin friction (shear resistance) widely compensate for each other (Fig. 11). The pile-raft interaction grows with the load, whereby the soil improvement due to the vibroflotation increases the direct load transfer via the raft into the ground. Thus, the top zone finally acts like a ”quasi-monolith”. The composite-like foundation system (raft + piles + improved ground) behaves as if its base were about 5.5 m below the raft. This can also be deduced from the gradients of the tangents of the curves in Figure 12: they are practically equal for the single pile state below the raft base on the one hand and the state of the group pile 5.5 m below the raft base on the other hand. The pile load has no effect on this basic behaviour. Furthermore, the thickness of the ”quasi-monolith” remains constant. Only the gradient of the curves decreases with the increasing loads. This means that the skin friction below the quasi-monolithic body increases with the load.

During the initial construction stage the external loads were nearly fully transferred from the raft into the subsoil. But the load transfer mechanism changed significantly after the spaces between the flat jacks and raft were injected step by step. This measure increasingly activated the piles, which during the quick construction of the high-rise structure, finally reached a maximum of about 75% (edge piles) to 95% (central piles) of the total design load, depending on the pile location and local load transfer from the high-rise structure. With ongoing settlement, the load portion directly running from

SLOVAK JOURNAL OF CIVIL ENGINEERING

2005/3 PAGES 1 – 12

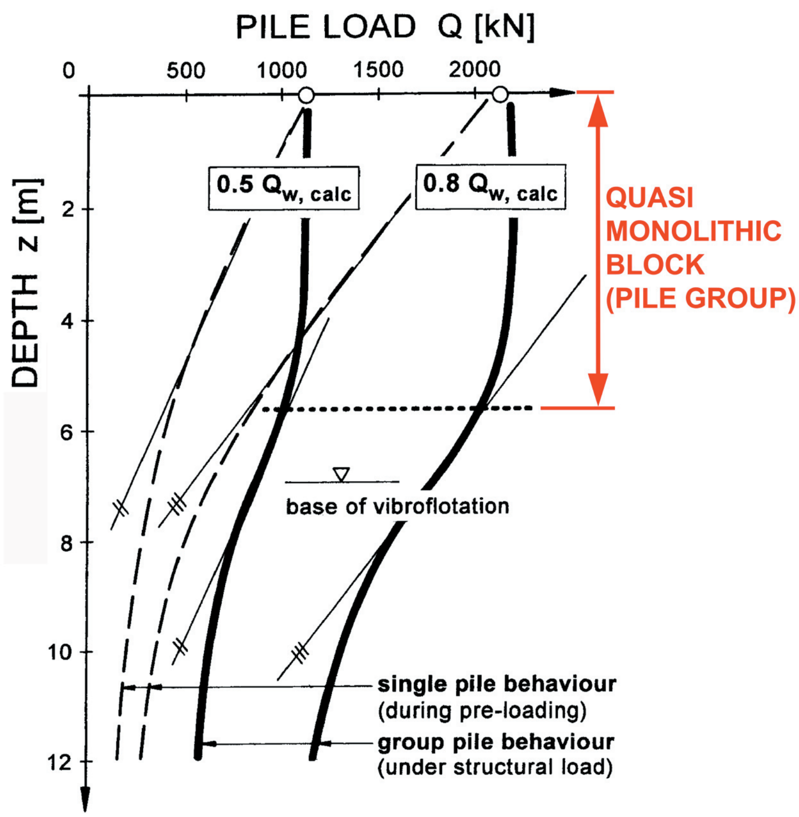

Fig. 12 *Normal pile force (Q) versus depth of the central measuring pile at different construction stages (0.5 and 0.8 $Q_{w,calc}$). At first, single pile behaviour during preloading, then group behaviour under construction conditions. Dotted line indicates the base of a widely "quasi-monolithic" body beneath the pile raft.*

the raft into the subsoil increased, thus causing a gradual decrease in the piles' load portion. The percentage of the load taken by the raft increased from originally 10 – 20 % to about 25 – 35 % of the specific total load.

SETTLEMENTS

The construction pits for the high-rise core and the lower structures were excavated simultaneously as one unit to a depth of 15 m below the surface. The excavation of 180,000 m^3 of soil caused a heave, which reached about 40 m below the level of the pit base: During excavation between 6 m to 15 m depth, ground heaving up to 30 mm was measured by a geodetic levelling survey and 60 m deep multiple extensometers (Fig. 13). From the recorded time-heave curve, an additional heave of nearly 10 mm could be deduced for the initial construction phase before the measurements started, thus resulting in about 40 mm total heave at the maximum. After casting the raft, the loads of the high-rise building were transferred very quickly into the ground: Because of the short construction period, two floors had to be erected weekly. A building height of 170 m was already reached after seven months. Therefore, the fresh concrete of the 2.2 m thick piled raft behaved very flexibly and underwent a trough-shaped settlement.

The total settlements were reached in the year 2002 (max. 38 mm), and they correspond fully with the prognosis. They can be widely considered as an elastic re-deformation of the ground after heaving during the deep pit excavation. The maximum differential settlement between the centre of the high-rise core structure and the edges of the building is $\Delta s \leq 23$ mm, and it occurred early, i.e. during the very flexible and statically non-critical phase of the construction.

Therefore, no compensation levelling was necessary during the

SLOVAK JOURNAL
OF
CIVIL ENGINEERING

2005/3 PAGES 1 — 12

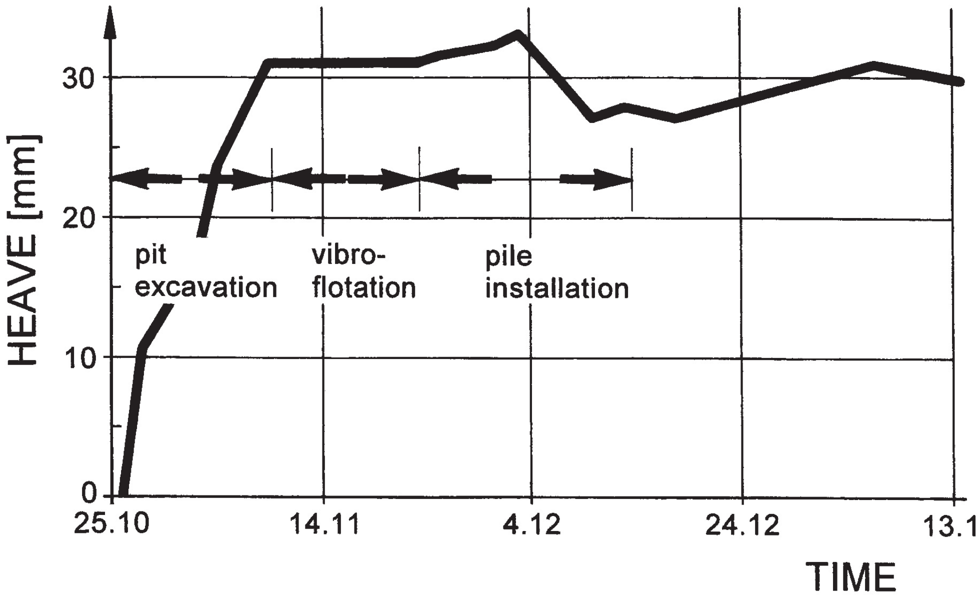

Fig. 13 *Ground heave due to the excavation of the 15 m deep construction pit (180,000 m³).*

construction of the high-rise structure, and the joints along the contact areas between the core tower and the adjacent 8-floor structures have remained closed. Furthermore, the convex settlement curve compensated for the concave deformation curve of the structural members within the high-rise structure above ground. The latter occurred because the compressive strains in the outer zone of the core were greater than those in the inner part. Consequently, the floors now exhibit horizontal slabs.

Figure 14 shows the settlement curves along the cross section of the building, which were derived from 20 measuring points between the centre point and the edge. It should be noted that there was exclusively a raft-soil contact at the beginning before the piles were gradually activated by injecting the open space on top of them. Figure 14 also indicates that the pile resistance was not mobilized from the beginning of the load transfer. At first, a direct load transfer from the raft into the ground occurred, and the portion of the pile load transfer was negligible. The piles were then increasingly activated by injecting the spacing between the pile head, flat jack, and raft in steps. The settlement trough developed due to the deep-seated overall ground deformation beneath the pile toes and not because of the differential pile behaviour.

The time-settlement behaviour is illustrated in Figure 15 and shows the mean value of the geodetic levelling and extensometer readings below the core structure. The effect of the groundwater lowering and raising within the cut-off walls of the excavation pit is also visible. The time-settlement curve exhibits increased flattening since the dead load of the building has been reached. This is typical of deep foundations in overconsolidated fine soils, where the settlements occur more quickly than in normally consolidated soil. Moreover the relevant subsoil zone contributing to settlements reaches not as deeply as in the case of normally consolidated soil.

The building was opened at the end of April 1999, and no measures to compensate for differential settlements have been required since then, though the subsequent construction of an adjacent building caused an additional settlement of up to 5 mm. Moreover, no joint between the high-rise core and lower structure has opened in the basement, which remains watertight. Proper serviceability in such cases is usually much more sensitive than structural stability.

COMPARISON OF DEEP FOUNDATIONS

In order to prove the reliability of geotechnical theories and the general application of test results to practice, in-situ measurements on construction sites and completed structures are essential. Figures 16 and 17 show some examples of deep foundations in tertiary sediments overlain by quaternary river deposits (Vienna). The tertiary layers are over-consolidated and consist of sandy to clayey silt (local silty sand and silty clay). The ground properties, of course, scatter in spite of the same geological genesis along the Danube

SLOVAK JOURNAL OF CIVIL ENGINEERING

2005/3 PAGES 1 — 12

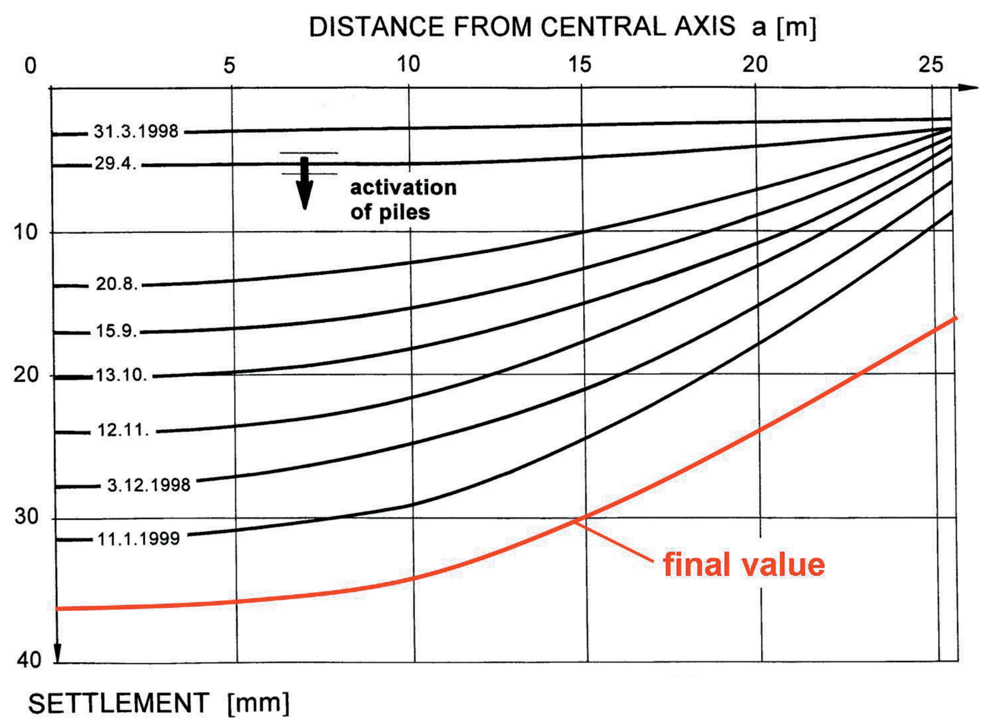

Fig. 14 *Settlement curves during construction of the Millennium Tower in Vienna. Gradual mobilization of pile resistance by injecting the spacing between pile heads, flat jacks, and raft.*

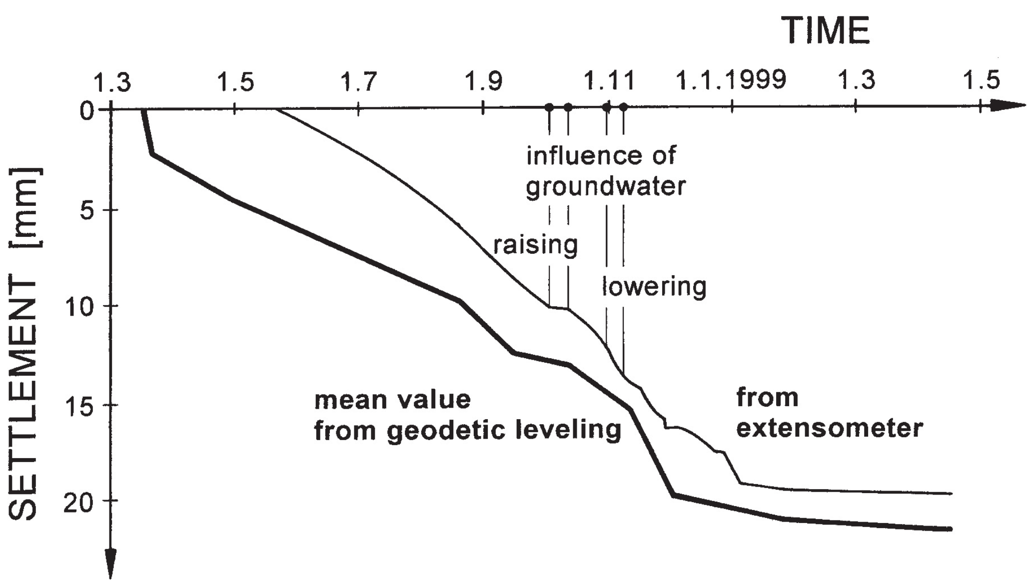

Fig. 15 *Partial time-settlement curves for typical measuring points of the Millennium Tower.*

SLOVAK JOURNAL OF CIVIL ENGINEERING

2005/3 PAGES 1 — 12

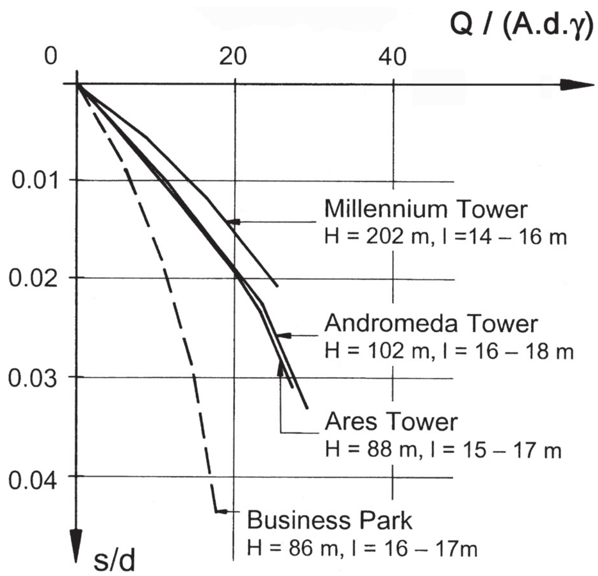

Fig. 16 *Normalized load-settlement behaviour of some high-rise buildings in Vienna, founded on bored or auger piles.*

s = settlement, d = pile diameter, Q = total load on the foundation, A = foundation area (horizontal sectional area within circumference of pile box), γ = density of soil, l = pile length, H = height of building

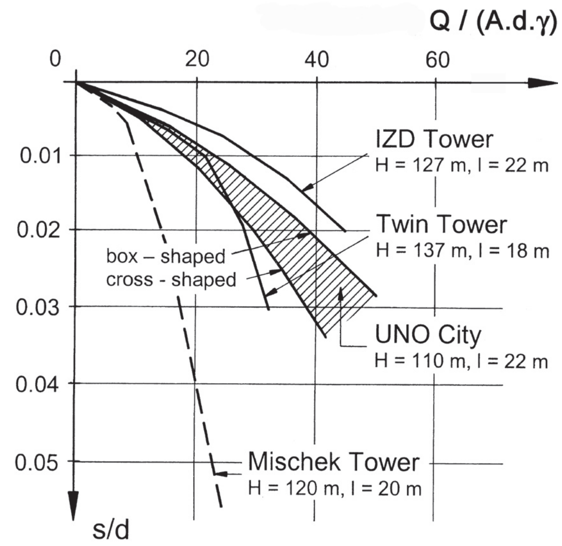

Fig. 17 *Normalized load-settlement behaviour of some high-rise buildings in Vienna founded on diaphragm walls.*

s = settlement, d = thickness of diaphragm wall, Q = total load on the foundation, A = foundation area (horizontal sectional area within circumference of diaphragm wall box), γ = density of soil, l = depth of diaphragm wall, H = height of building

River in Vienna and nearby. Nevertheless, the site conditions of the structures can be fairly compared. The deep foundation elements were large diameter bored or auger piles (d = 0.9 to 1.2 m) or diaphragm walls (thickness, d = 0.6 to 0.8 m).

The data are plotted in dimensionless diagrams, because normalized graphs enable a direct comparison with different buildings and model tests. The maximum total settlements of high-rise buildings in Vienna vary between 3.8 cm (Millennium Tower) to nearly 10 cm (Mischek Tower). Most values lie between 5 cm and 7 cm.

Figure 16 shows the results from some high-rise buildings in Vienna. All buildings have deep basements, i.e. the top of the foundation clearly lies below the original surface, and the foundations are below groundwater level. The pile lengths are of a similar magnitude; the differences are only local, depending on statical or structural requirements. The diagram illustrates very clearly the advantage of piled raft foundations over conventional pile foundations (Business Park). The optimum behaviour of the Millennium Tower can be explained by pile preloading and improvement of the soil in the top zone of the foundation: the loose sandy gravel overlying the tertiary silt was densified by vibroflotation.

Figure 17 shows the normalized load-settlement curves of several box foundations with diaphragm walls. A comparison underlines the following interacting factors of influence:

- Length (depth) of wall elements: Widely superimposed by other factors, because l = 18 – 22 m is roughly of a similar magnitude, hence of secondary effect in this special case.
- Level of foundation head: The high-rise building without a deep basement (Mischek Tower) settled more than the others.
- Method of installation: Installation from a higher working level improves the bearing-deformation behaviour of diaphragm walls (e.g. IZD-Tower); this is also the case for bored or auger piles. The positive effect of an uncast pile length or uncast diaphragm wall depth could be observed at many construction sites. This is achieved because the soil along the top zone of a deep foundation is less disturbed during the installation procedure.
- Effect of geological overconsolidation: The Twin Tower is situated in an area of less overconsolidation than the other buildings.
- Arrangement of diaphragm walls: The buildings of the UNO-City Vienna are founded on box-shaped as well as cross-shaped wall elements. The latter have settled more, because the composite effect between the concrete panels (single barrettes, cross-shaped) and soil is clearly smaller than for box foundations.

SLOVAK JOURNAL OF CIVIL ENGINEERING

2005/3 PAGES 1 — 12

CONCLUSIONS

Comprehensive in-situ pile testing has confirmed that the load-settlement behaviour of the individual piles of a structure usually differs relatively widely during the first loading. Consequently, preloading in connection with cyclic load hystereses of piles represents an effective method to reduce the total and differential settlements of high-rise buildings and/or statically sensitive structures, e.g. continuous girder bridges. This can be performed during an early stage of construction by using a piled raft or capping structure as a counterweight/ kentledge. Preloading should be carried out on each pile, and it should be conducted in such a way that loading-unloading-reloading cycles are applied until all load-settlement curves of the piles are parallel along the statically relevant load sections. Preloading should exceed the calculated working load (the design load for service conditions) of the piles by at least 20 %. Usually, two to three hysteresis loops of load cycles are sufficient.

This innovative procedure represents an analogy to the control and acceptance tests of prestressed ground anchors, but it is improved by cyclic loading of each element and therefore simultaneously involves quality assessment and improvement. Consequently, it could be a future technique for in-situ pile testing/checking and (differential) settlement reduction for statically sensitive buildings or structures with a sensitive architecture (e.g. special glass facades). The specific increase in quality is by far higher than the increase in cost. Building activities are not hindered because the preloading procedure can be performed completely independently of other construction work.

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