PREDICTION OF THE PILE BEHAVIOUR UNDER DYNAMIC LOADING USING EMBEDDED STRAIN SENSOR TECHNOLOGY

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abstract

A standard dynamic loading test of the pile was performed on the highway section Slivnica – Hajdina near Maribor, Slovenia. Parallel to standard testing procedures the new monitoring technology based on specially developed strain sensors installed inside the pile body along the pile axis was introduced. On the basis of the measured results the normal strains along the pile axis were measured. Taking into consideration the elastic modulus of the concrete the normal stresses in the axial direction of the pile were also calculated and afterwards the shear stresses along the pile shaft have been estimated as well as the normal stresses below the pile toe. The estimation was made by considering a constant value for the pile diameter. The measured results were also compared with the computer simulation of the pile and the soil behaviour during all the successive test phases. The strain measurements inside the pile body during the standard dynamic loading test in present case did not have the purpose of developing an alternative method of pile loading tests. It gave in the first place the possibility of a closer look at the strains and stresses of the most unapproachable parts of different types of concrete structure elements especially piles and other types of deep foundations. The presented monitoring technology proved itself as a very accurate and consistent.

1 INTRODUCTION

The bearing capacity and settlement of vertical loaded pile can be estimated using different methods [2, 6]. In order to identify the pile behaviour and its interaction with the surrounding soil layers, knowledge of the state of the strains along the pile axis inside the pile body is of essential importance. To make this possible, a chain of measurement points has to be included outside the pile structure and it should be fixed to the appropriate position on the pile reinforcement before the reinforcement is placed in the pile pit. After that the standard procedure for concreting the pile must be executed. The first opportunity for testing this idea was an estimation of the testing pile shaft's resistance, as described in the paper Štrukelj et. al. [8]. The basis of this estimation was the measurement of the normal strains of the pile in its axial direction at measuring points distributed over equal distances along the pile axis. These strains are proportional to the axial forces in the pile. When the course of the axial force along the pile axis is known, the resistance of the pile shaft can be estimated. The measurement points were distributed over equal distances of 1.0 m, starting 0.5 m above the pile toe. Since the strain gauges, electrical contacts and communication cables are very sensitive, any moisture

and mechanical loading could be very harmful to their performance. Therefore, the measuring points were protected with special care. To ensure the mechanical protection of the chain of strain gauges together with the communication cables the whole measuring system was built inside a steel tube made of two standard C-profiles. The interior surface of the steel tube also served as the grounding surface to which the strain gages could be glued. For transportation reasons the tube was made of three segments that could be easily put together. The mutual connection of the segments was ensured by a special system of joints. The tubes external surface was degreased and made rough to ensure the adhesion with the concrete. Each strain gauge inside the tube was protected against the moisture. When the measuring tube was finished, it was connected with the pile reinforcement and put into the pit that was prepared for the pile concreting. The measurement system fulfilled the expectations and the measured results were accurate and stable. The only disadvantage of this measurement system was its non-flexibility. It could only be used to build the measurement chains where the measurement points are placed along one line and oriented in the same direction. The loading test included seven loading cases. Each represented the drop of a steel weight from a different height to the top of the test pile and the second and third hits after the first and second repulsions [9].

The location of the testing pile was selected on a construction site of the highway section Slivnica – Hajdina in Slovenia. The geological conditions of the testing pile's location and the positions of the nine sensors are shown in Fig.1, and described in Section 2.1. In the following sections the measuring equipment, its installation, the performance of dynamic loading test, and finally the evaluation of the measurement results is presented. In the last part of the paper the comparison of the measured results with results of numerical axis-symmetry analyses using the finite-element method (FEM) is presented.

2 PRELIMINARY WORKS ON THE TESTING SITE

2.1 GEOLOGICAL FIELD AND LABORATORY INVESTIGATIONS

Geological conditions of the wider location of the testing site were acquired from the geotechnical report for the design of a highway crossover foundation [7]. The strength and deformability parameters were defined on the basis of field investigations by a standard penetra-

tion test and probe measurements, as well as by laboratory testing of the samples taken by sounds of depth up to 25 m from seven different locations. The region belongs to the eastern part of the River Drava field that is mainly a plain area with only small differences in height. The relief evolution is based on the accumulation river-denudation with river influences in the past, which have deposited a layer, more than 10 m thick, of gravel GP, sandy gravel GP-GM, and some lenses of sandy clay CL with boulders over the Miocene base of sandy marl.

On the location of the testing pile one additional sounding to the depth of 15.0 m was performed. The strength and deformability parameters at some depths of this additional sound were defined on the basis of field investigations by standard dynamic penetration tests, and also on the basis of the laboratory testing of samples. The ground water level in this sounding was encountered at approximately 3.30 m below the surface. The cross-section of the ground space with the pile (Fig. 1.) is composed of a thin, 1.0 m layer of embankment that was built on an original space of eight characteristic layers of different thicknesses.

Fig. 1 shows the geological conditions of the testing pile's location and the positions of the nine sensors. Table 1 presents the strength parameters and the classification (ϕ is the internal friction of the soil, c is the cohesion of the soil, and E_{oed} is the oedometer elasticity modulus of the soil) of the layers presented in Fig. 1, which were determined on the basis of laboratory and field testing on additional sound samples.

2.2 MEASURING EQUIPMENT

Besides the standard equipment that is needed for an estimation of the bearing capacity of the pile on the basis of a dynamical loading test [1], the additional specially produced strain sensors were built into the pile body. They were placed at equal distances of 2.0 m along the pile axis, starting at 1.5 m from the pile toe. The patented sensor design used for this purpose is very efficient, easy to build in, and robust enough to stand the water pressure and all the possible mechanical burdens during the pile concreting. The final solution for the strain sensor was to build a complete strain sensor for a single measurement point for all the layers of protection coatings and wiring. Such a sensor can be placed in the desired position in a very short time. It is insensitive to moisture and dust and its vital parts are very well protected against mechanical damage. The basis for such a sensor is a standard reinforcement bar of length about 150 cm and diameter 16 mm. In the
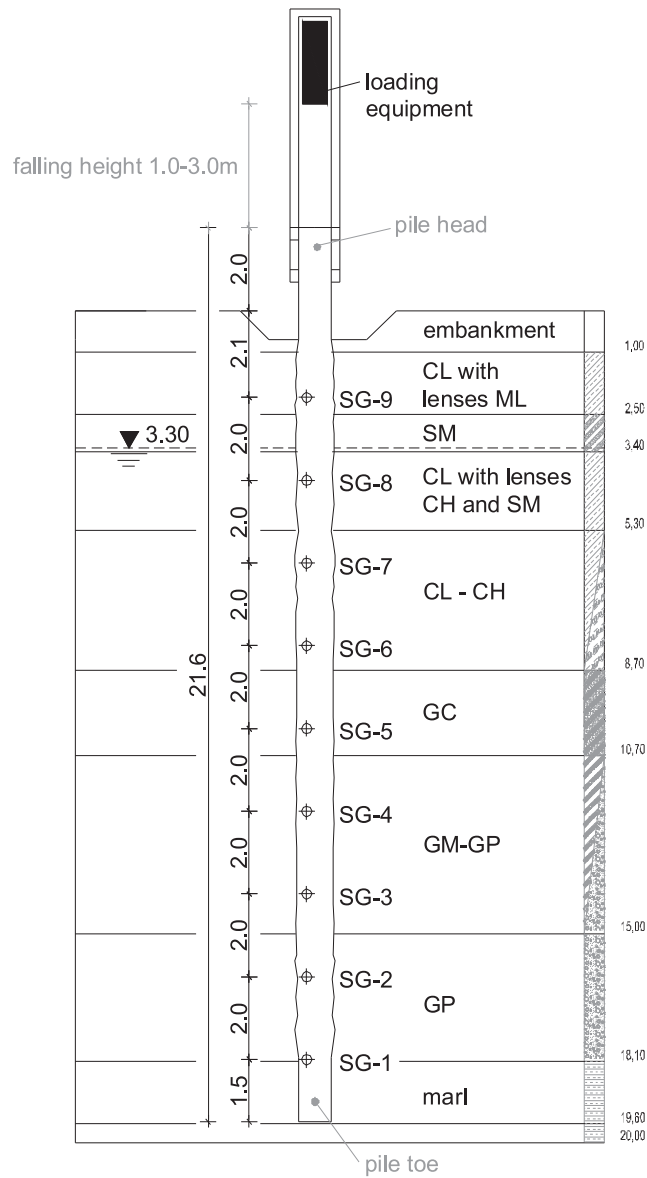
Figure 1. Section of the testing site with the disposition of the soil layers and the measuring points on the pile.

Table 1. The strength parameters of the soil layers.

depth [m]	ϕ [°]	c [kPa]	E_{oed} [kPa]	soil classification
0.0-1.0	36.0	0.0	40000	embankment
1.0-2.5	23.0	20.0	1800	CL with lenses ML
2.5-3.4	25.4	0.0	2700	SM
3.4-5.3	24.7	20.0	19000	CL with lenses CH and SM
5.3-8.7	24.7	21.8	14000	CL-CH
8.7-10.7	33.0	0.0	11700	GC
10.7-15.0	38.8	0.0	43200	GM-GP
15.0-18.0	34.7	0.0	20700	GP
18.0	30.0	300.0	450000	marl

middle of the reinforcement bar its surface is grinded on one or both sides, depending on the number of strain gages that should be installed. These strain gauges can be connected in a full, half, double-quarter or quarter Wheatstone bridge [4, 5]. The connecting cable is protected by a polyethylene tube and fixed to the reinforcement bar. The protection coating consists of two layers of polyurethane varnish, a layer of special silicone putty and a layer of permanently plastic sealant putty coated by aluminium foil. This combination of protection was tested and remained waterproof even at 30 m under the surface of water. The final layer represents the physical protection and can be made of polyethylene

tube (Fig. 2) when the dimensions of the measured concrete elements are not too small to be significantly weakened by the built-in sensor. Otherwise, the physical protection of the measurement area can be made of cement mortar with the addition of an acrylic emulsion. This type of strain sensor proved to be very reliable, easy to build and cost effective, and can also be used together with the appropriate equipment and software [11] for the purposes of monitoring the other parts of structures. The successive phases of the sensors' preparation and their placing in the planned positions of the pile reinforcement are shown in Figs. 2–5.
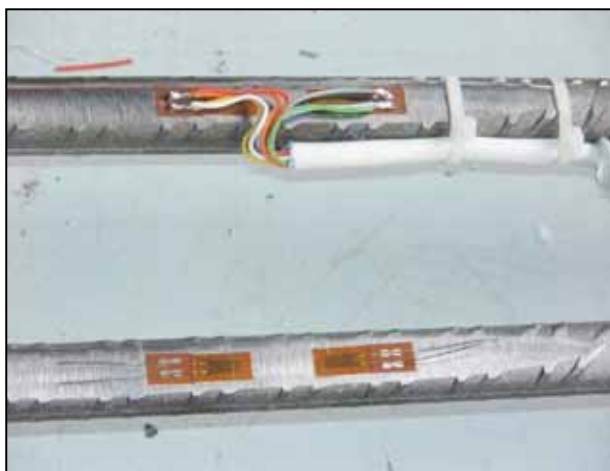

Figure 2. The first phase of sensor preparation: applying the strain gages and wiring.

Figure 3. Finished sensors prepared for installation in the pile reinforcement.

Figure 4. The strain sensor fixed to the pile reinforcement.

Figure 5. Placing the pile reinforcement equipped with nine strain sensors into the pile pit.

2.3 CONSTRUCTION OF THE TESTING PILE

The testing pile was constructed as a bored reinforced concrete pile [2] of 80 cm in diameter. The final length of the testing pile was 21.60 m. The length of the underground part of the pile was 19.60 m. As the thickness of the sandy clay, sandy gravel and gravel layer was about 18.0 m, the testing pile was embedded about 2.0 m into the marl base. After the pile was cleaned up to the level of the working area a 2.0 m long reinforced lengthening piece was additionally concreted on the top of the pile head (Fig. 6).

2.4 INSTALLATION OF THE DYNAMIC LOADING EQUIPMENT

The gravitational weight was applied for loading, which was mounted on a steel guidance with an inner diameter of 80 cm and guiders with a length of more than 5.0 m (Fig. 7). The weight could fall from a height of 3.0 m. The assembly and lifting of the weight were managed by a 40 tons portable derrick. The dropping of the weight was performed with full gravity falling of a steel ballast weighing 10.5 tons on the head of the pile. The steel ballast was built in a 3.5 tons heavy steel frame (Fig. 7). At the top of the pile between the concrete surface and the falling steel ballast two wooden boards were placed to prevent the concrete being crushed.

Figure 6. The top of the testing pile before the installation of the loading equipment.

Figure 7. The testing site during the dynamic load test.

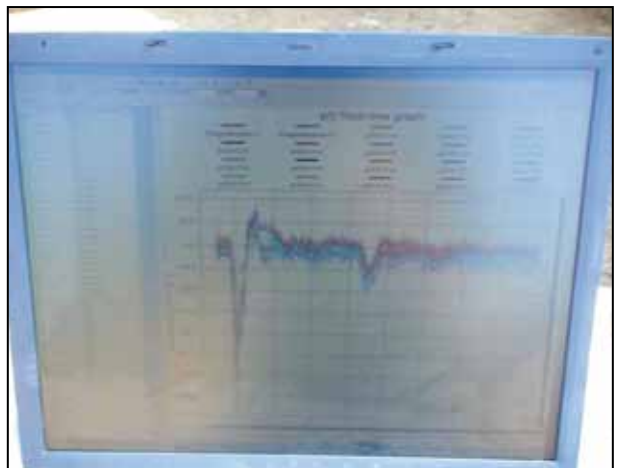Figure 8. The configuration of the measurement equipment during the loading test.

Figure 9. Measured results displayed online on the computer screen during the loading test.

3 THE PERFORMANCE OF THE DYNAMIC LOADING TEST

During the test the steel ballast with a mass of 10,500 kg was dropped seven times to the head of the pile from different heights (from 1.0 m to 3.0 m). The exact data about the height of the gravity falling of the steel ballast during each loading phase are presented in Table 2.

During the test all the sensors built inside the pile body were connected to the data-acquisition unit (Fig. 8). Because of the explicitly dynamic nature of the measured events the chosen loading rate was 2400 Hz. The additional certainty of the measured results was achieved by installing two separate strain gauges in each sensor. Each strain gauge on each measuring point was connected independently to the data acquisition and the measured result in the individual measuring point is believed to be accurate when both strain gauges from each sensor give the same result. The measured results were stored on a computer hard disc during the measurement and they were also displayed online on the computer screen (Fig. 9) during the measurement procedure [9].

4 EVALUATION OF THE MEASUREMENT RESULTS

The review of the measured strains versus time for the three characteristic loading cases (1st, 4th and 7th) is presented in Fig. 10. From the sequence of three loading cases it can be seen that the amplitude of the strains increases with the increased height of dropping the steel ballast. The designated area of the last of three diagrams in Fig. 10 represents the most interesting part of the strain distribution during the primary hit of the steel ballast in the seventh loading case. This part of the measured signal is shown in the detailed view in Fig. 11 and represents the peak of the measured values during the entire test. From the time histories of the strains recorded during each loading case it is clear that the steel ballast after first hit to the top of the pile repulsed. The amplitude of the measured strains after the second hit after the first repulsion is about 25 percent of the strain amplitude of the primary hit. During each loading case the strains were recorded until the system had completely come to rest. All the measured signals were precisely analyzed afterwards. The result of this analysis is shown in Fig. 12, which represents the review of the peak strain values at all the measurement points for the first, second and third hits of the dropping mass for all the loading cases. Considering the value of the elasticity modulus for the pile's concrete is 34 GPa; the peak values of the normal stresses in the pile's axial direction (Fig. 13) were evaluated from the strains given in Fig. 12. After the equilibrium condition was used, and it was considered that the pile diameter remained constant along its whole length, the shear stresses along the pile shaft were also evaluated for all the loading cases. The results of this evaluation are presented in Fig. 14. The measurement results gave the values of the remaining strains after each loading case as well. The integrals of these values along the whole pile axis gave the values for the pile contraction for each loading case. Since the settlements of the pile head were geodetically measured after performing each loading case, the pile-toe settlements could also be evaluated. These values are presented in Table 2.

Table 2. The evaluation of the pile-toe settlements.

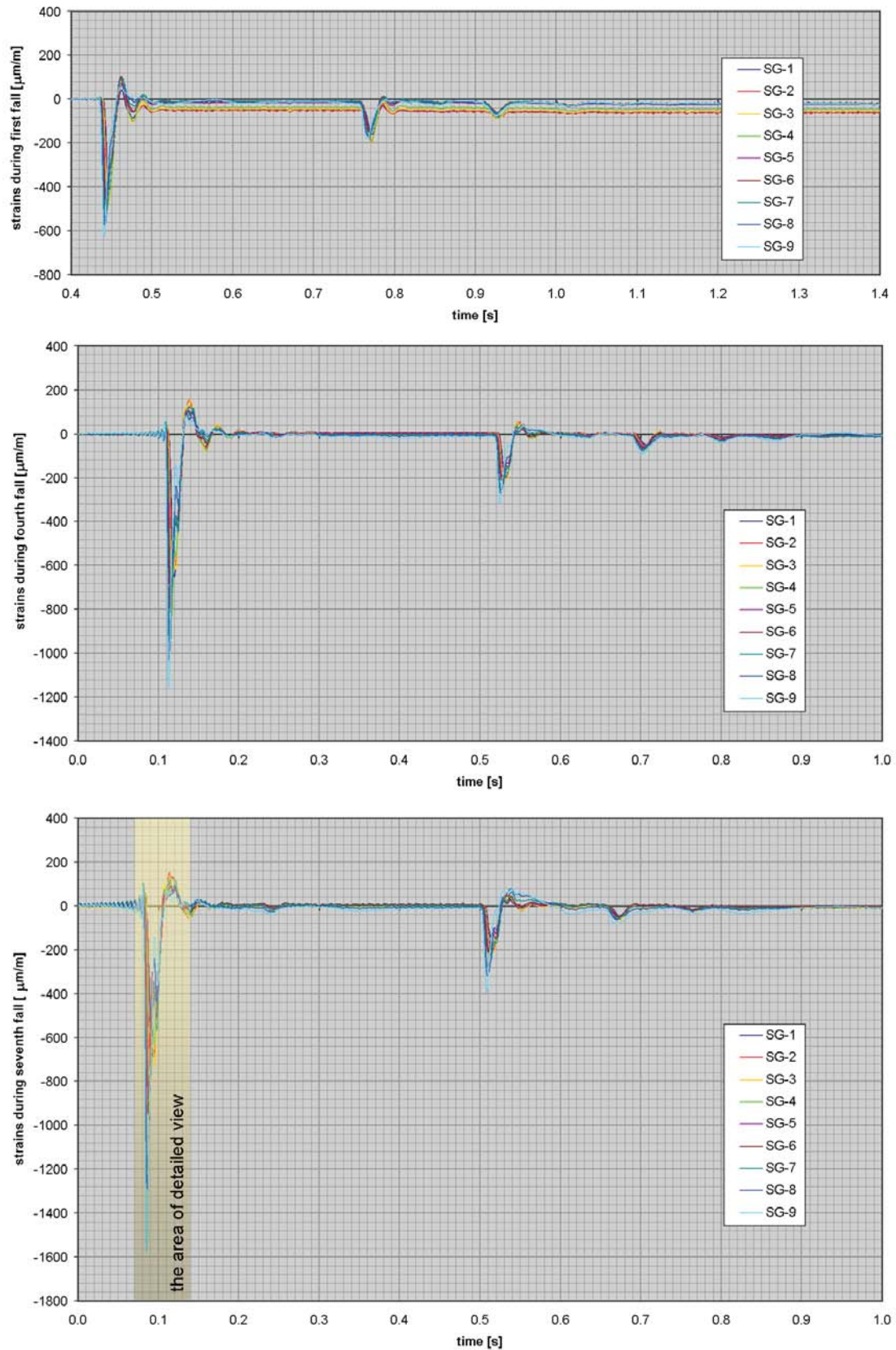| number of loading case	height of gravity falling [m]	pile contraction [mm]	cumulative contraction [mm]	pile-head settlements [mm]	cumulative pile-head settlements [mm]	pile-toe settlements [mm]
1	1.0	0.65	0.65	4.00	4.00	3.35
2	1.0	0.04	0.70	2.00	6.00	5.30
3	1.5	0.11	0.81	1.50	7.50	6.69
4	2.0	0.17	0.97	1.50	9.00	8.03
5	2.5	0.14	1.11	4.50	13.50	12.39
6	2.5	0.23	1.35	3.00	16.50	15.15
7	3.0	0.15	1.49	1.50	18.00	16.51

Figure 10. Measured strain signal recorded after the first, fourth and seventh (the last) falls of the dropping mass. In all the signals the strain peaks after the first hit of the dropping mass as well as the second and third hits after both repulsions can be clearly seen.
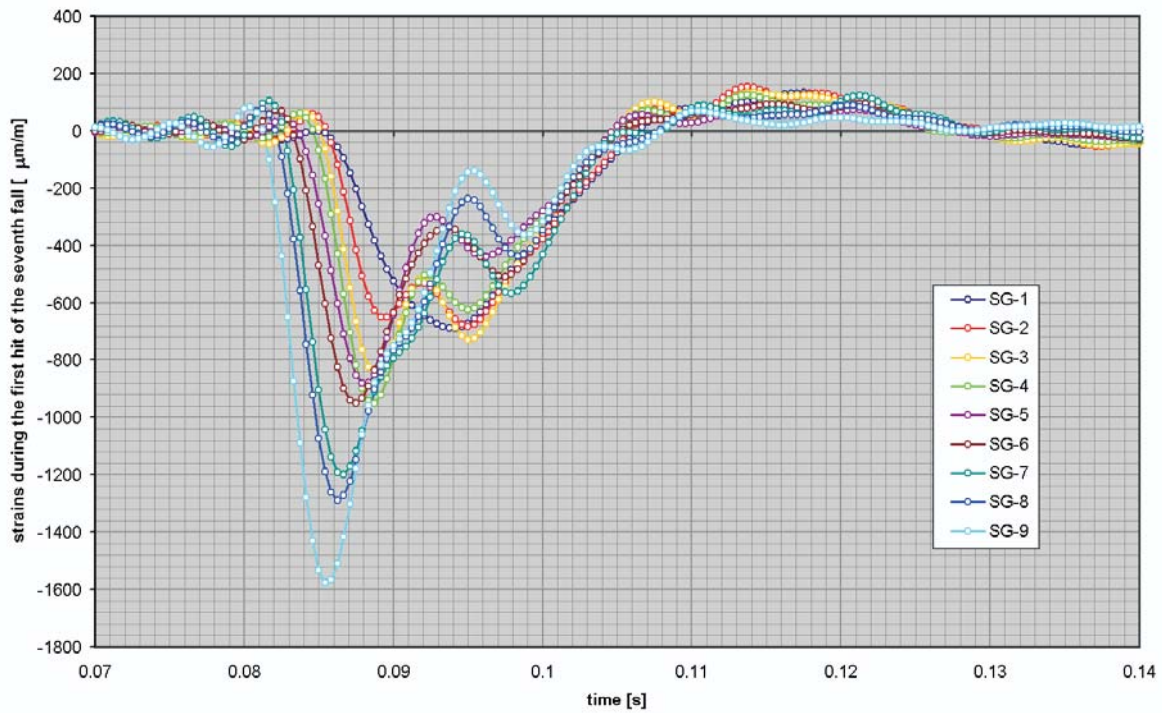
Figure 11. Detailed view of the first peak of the strains in the measured signal of the seventh fall of the dropping mass.
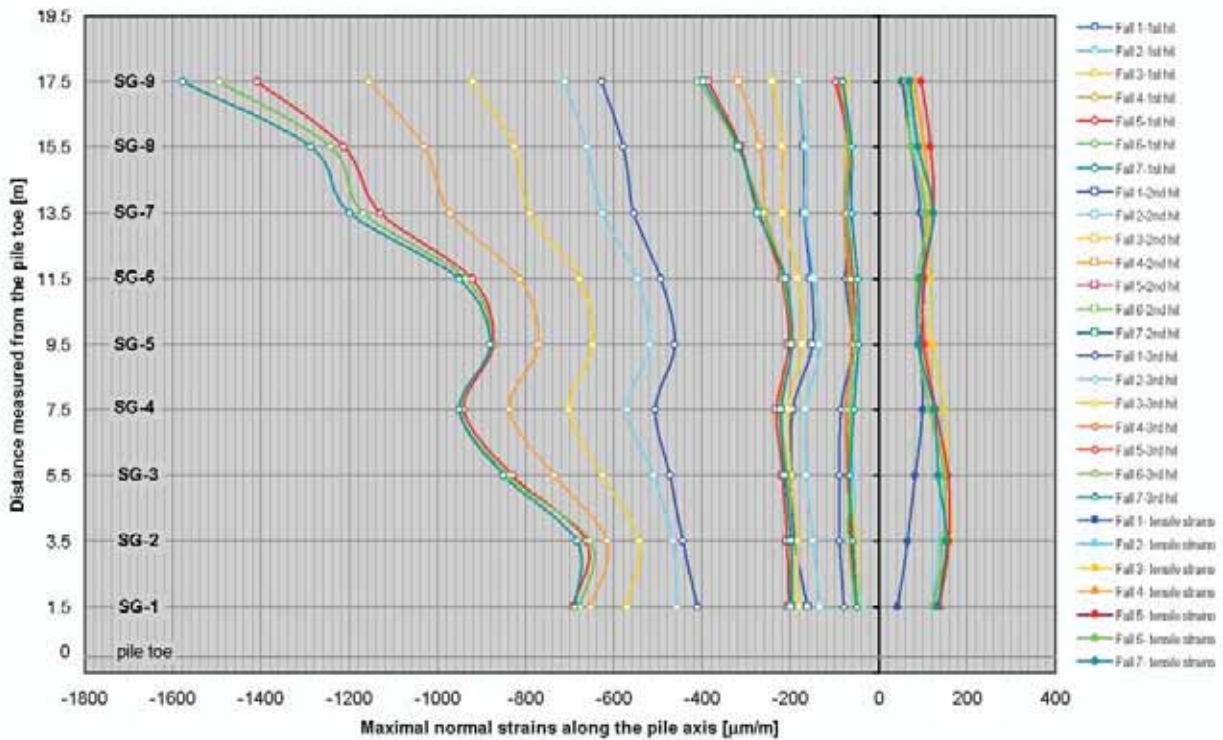

Figure 12. The review of the peak values of the measured results where the peak strain values at all the measurement points for the first, second and third hits of the dropping mass for all the loading cases are shown. In the figure the maximum tensile strains occurring during the hit-wave propagation along the pile are also presented.
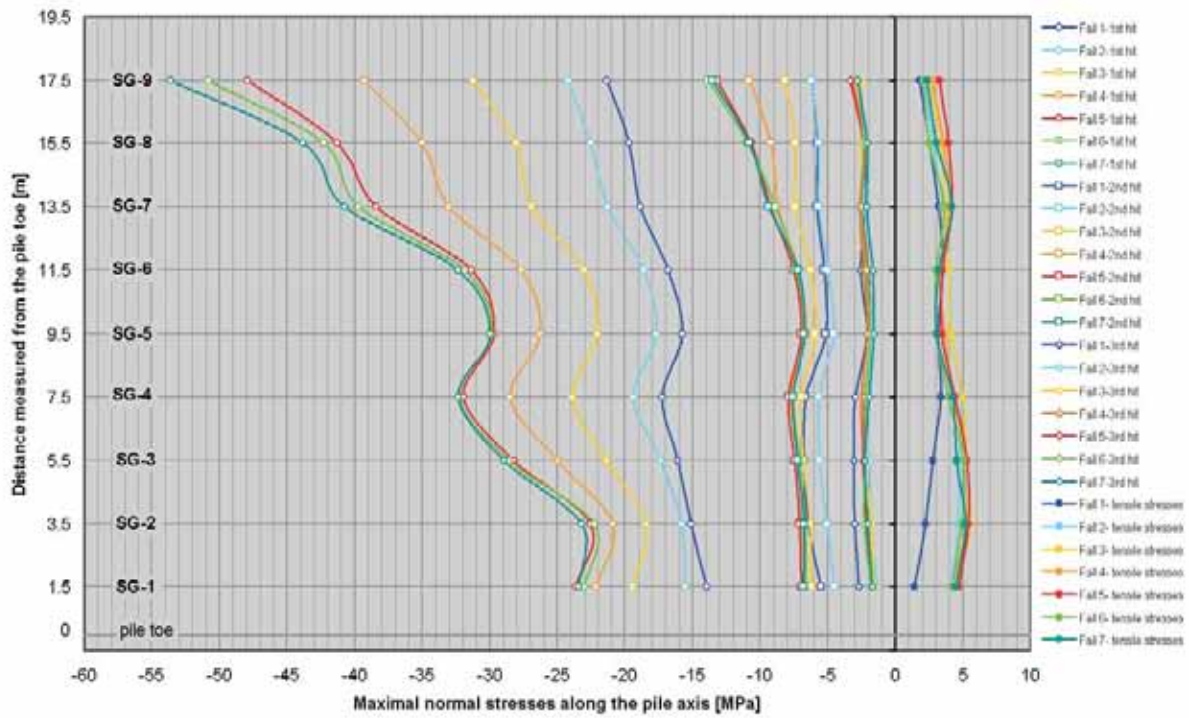
Figure 13. A review of the peak values of the normal stresses in the pile's axial direction calculated from the measured peak strain values at all the measurement points for the first, second and third hits of the dropping mass for all the loading cases.
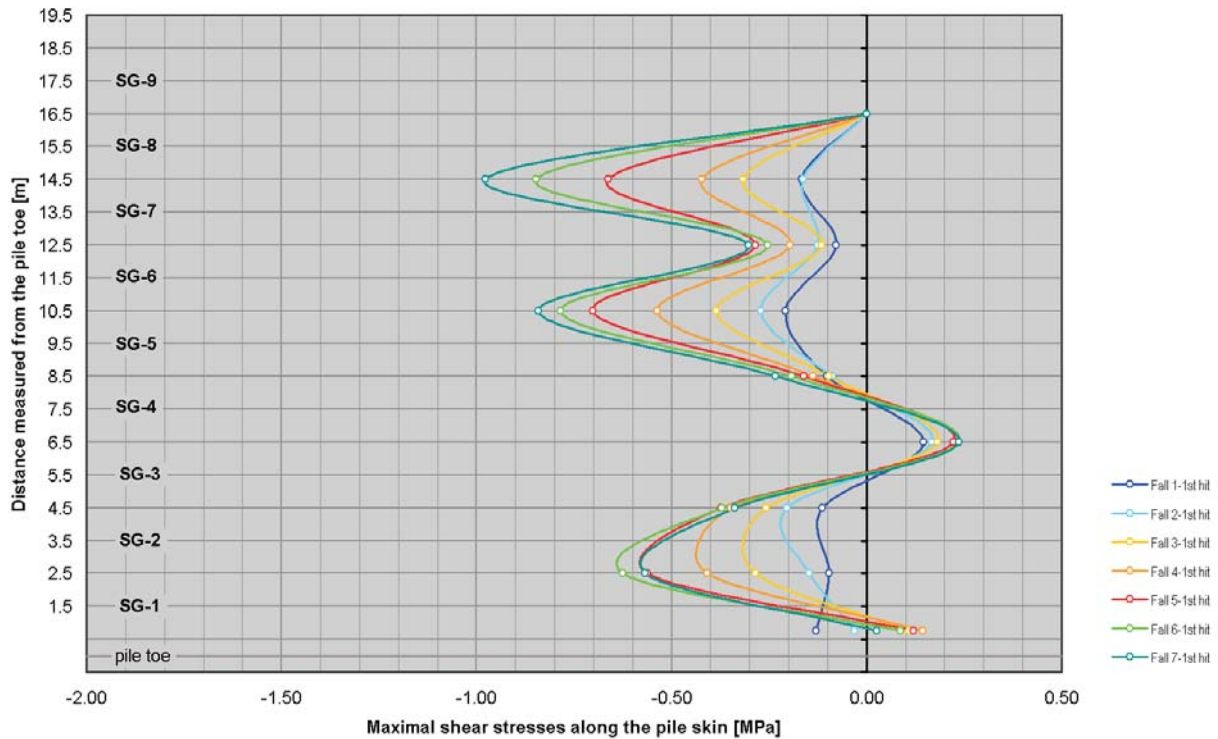

Figure 14. Curves of the peak values of the shear stresses on the pile shaft for all seven loading cases. The continuous lines were calculated from the measured results, whereas the dashed lines are extrapolated.

5 FINITE-ELEMENT ANALYSIS

The results of field investigations were compared with a set of numerical analyses using the finite-element method (FEM). Axis-symmetric analyses of the interaction between a bored reinforced pile loaded with vertical dynamic loading and the ground were performed for this purpose.

The analyses considered a 20.0 m long pile that is placed in a length of 18.0 m in eight different soil layers and below this to a depth of 2.0 m in a marl base. The ground water level is 3.30 m under the ground level.

The reinforced concrete of the pile was considered to be linear elastic with a Young's modulus $E = 34.0$ GPa, a Poisson's ratio $v = 0.2$ and a unit weight $\gamma = 25$ kN/m³.

The strength properties of the ground (see Table 1) were determined on the basis of laboratory and field testing results of additional sound samples. This soil half-space was designed in numerical analyses by Hardening-Soil material model with isotropic hardening. This model considers nonlinear elastic hyperbolic dependence between stresses and strains; it enables the consideration

of increasing soil yielding as a function of ground stresses, dilatation and cap yield surface, and is not based on theory of plasticity [3]. The parameters in the elasto-plastic Hardening-Soil model $E_{oed}^{ref} = E_{50}^{ref}$ and $E_{ur}^{ref} = 3\,E_{50}^{ref}$ where E_{oed}^{ref} is the tangent stiffness for the primary oedometer loading at the reference pressure, and E_{ur}^{ref} is the unloading/reloading stiffness [3]. The cross-section of the analyzed model and the corresponding finite-element mesh is presented in Fig.15.

Fig. 16 presents the absolute displacement after each loading phase of seven blows to the point A (at the pile head) and the point B (at the pile toe). Each loading phase is combined with two separate dynamic stages, where the first stage performs the dynamic force or stroke, and the second stage performs the relaxation time of 0.2 sec. This time was the relaxation period of the pile displacement also at the actual state according to the measurement results (see Fig. 10). The loading for the simulated case was calculated from the measured peak stress value shown in Fig.13. The results of the FEM analysis; the vectors of the vertical displacements after the last blow where $u_{max} = 22.70$ mm at the pile head and $u_{max} = 19.80$ mm at the pile toe (see Fig. 15).
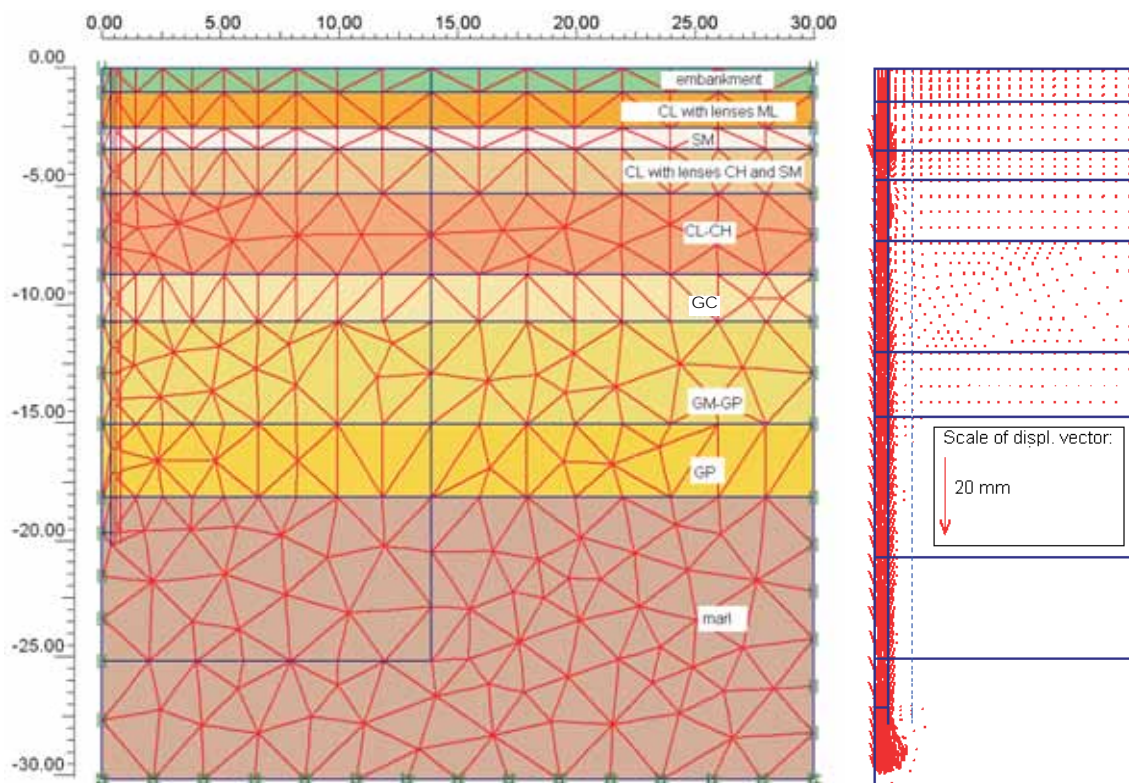

Figure 15. Cross-section of the model with finite-element mesh and soil layers (Fig. left) and vectors of displacements after the last blow (Fig. right).
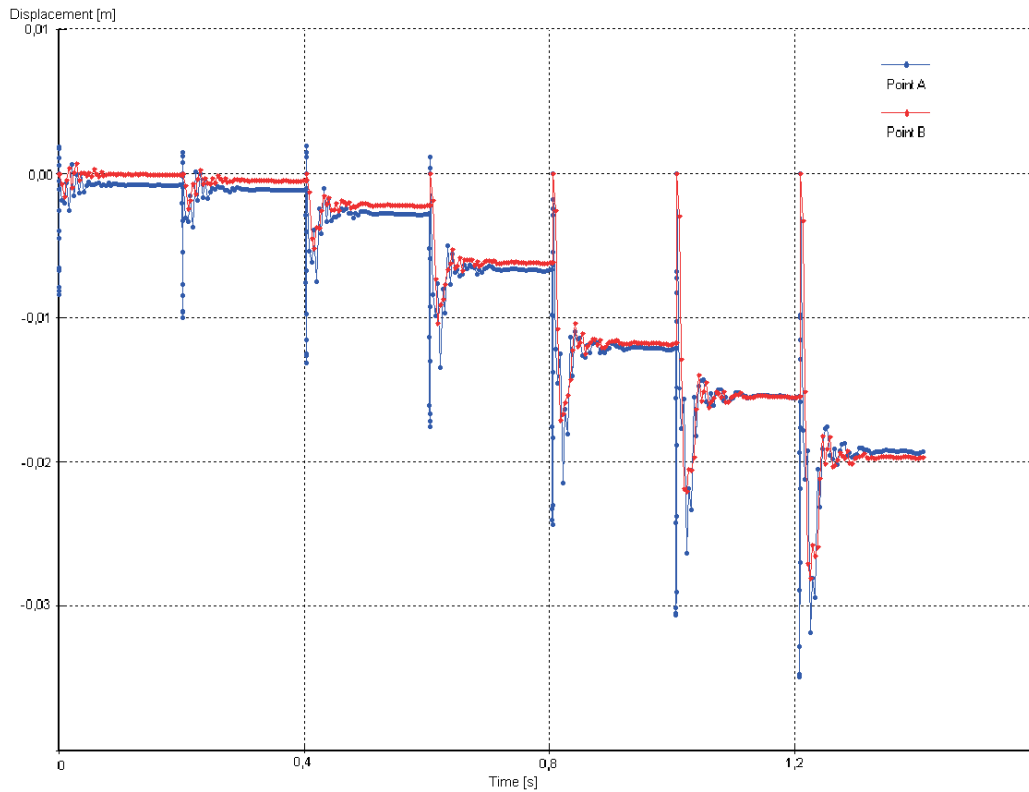
Figure 16. Absolute displacement after each blow of the point A (at the pile head) and point B (at the pile toe).

6 COMPARISON OF THE MEASUREMENT RESULTS WITH THE FINITE-ELEMENT ANALYSIS RESULTS

The finite-element analyses of the axis-symmetrical FE model of pile and surrounding soil gave the results that are comparable with the results of methods based on measurements.

Fig. 17 shows the relation between the vertical loading and the movements of the pile obtained with the analysis on the axis-symmetrical model compared to the measured values, and to the results of CAPWAP (Case Pile Wave Analysis Program) which separates static and damping soil characteristics and also allows an estimation of the side shear distribution and the pile's end bearing. CAPWAP is based on the wave equation model, which analyses the pile as a series of elastic segments and the soil as a series of elasto-plastic elements with damping characteristics, where the stiffness represents the static soil resistance and the damping represents the dynamic soil resistance. Typically the pile top force and velocity measurements acquired under high strain

hammer impacts can be analyzed utilizing the signal matching procedure yielding forces and velocities over time and along the pile length. [1, 10].
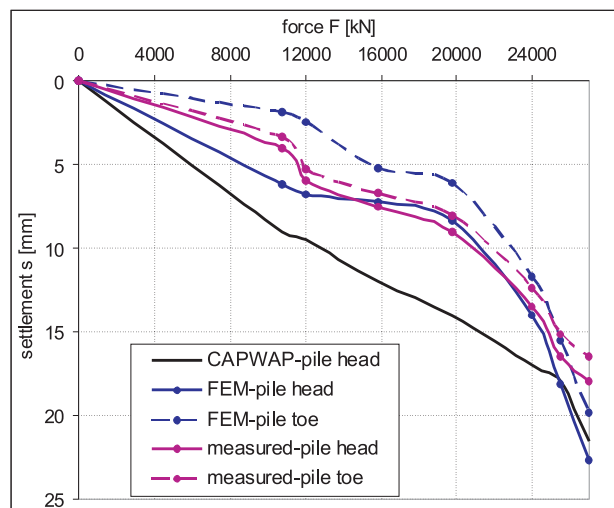

Figure 17. Comparison of analyzed (FEM) and measured values of the vertical load (F) versus the settlement (s) of the pile head and the pile toe.

The cumulative contraction of the pile after each blow is comparable. The difference between the pile-head and pile-toe settlement during the first three blows is higher, while after the third blow the difference between the measured and the analyzed results is decreasing. It can be concluded that the results of the settlements of the field measurements and the FEM analysis are within a comparable range.

7 CONCLUSIONS

The present paper describes the development and application of a new measuring technology that proved itself to be very useful for investigating the behaviour of piles and also other embedded structures as well as the surrounding soil loaded by different kinds of static and dynamic loads. The presented results show that the state of the strains inside the pile body can be obtained in a very accurate and cost-efficient way even for events that show an extremely dynamic nature. In addition, using the described measurement method with the appropriate distribution of measurement points and measurement directions, the most complex states of strain can be followed. In our paper the presented example shows that the extreme values of the measured strains, and the calculated stresses based on these strains, are significantly higher than the values given in an official report of the standard dynamic pile test. However, it should be considered that the values obtained on the basis of the strain measurements are the peak values of the dynamic response and not the reduced values used for the simulation of the static behaviour of the pile on the basis of a dynamic test. Additionally, it should be stated that the purpose of the demonstrated measurement technology is in the first place to create the possibility of a closer look at the behaviour of the most unapproachable parts of essential construction elements, like piles and other types of deep foundations. The parallel measurements of the strains inside the pile body during the standard dynamic loading test therefore did not have the purpose of developing an alternative measuring procedure for the dynamic loading test. In this case the standard loading test served only as an opportunity for testing the applicability, accuracy and the consistency of the new measurement technology.

Even though the conclusions were written before, the results of the field investigations were compared with a set of numerical analyses using the finite-element method (FEM). In the presented computer simulation the model with the axial symmetry was used for the interaction between a bored, reinforced pile loaded dynamically in the vertical direction. The comparison of the results obtained by both measurement methods and the computer simulation is acceptable. In addition, the further analysis of the results shows that the agreement of the calculated and measured values is even better after using some parameters for the soil model employed in a computer simulation and obtained from the analysis of the measured results.

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