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Selected modeling problems of monopile foundations used in the energy industry

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Abstract: The paper presents selected practical aspects of the construction of monopile foundations used in the onshore power sector and analysis the effect of these solutions on the design of such structures. The basis for the evaluation of load capacity of such foundations was briefly discussed and the key factors that determine the ability to transfer loads from the structure to the ground were indicated. The main problem presented in the paper was to assess the impact of the reinforced concrete stay-in-place formwork piles built using the well technology. The considerations were supplemented with a computational example illustrating the thesis presented in the paper. A positive effect of prefabricated formwork rings on the load capacity of the foundation was demonstrated.

Keywords: foundation of the poles; monopiles foundations; FE modeling

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Introduction

Supporting structures for overhead power lines (OHL) must now be designed in Poland according to PN-EN 50341-1 (2013) and PN-EN 50341-2-22 (2016). These standards define both the magnitude of impacts and loads necessary for the design of overhead power lines, the relevant load combinations, and the principles for the calculation and design of their supporting structures. In the case of medium (up to ca. 45 kV) and higher-voltage lines, steel structures are the dominant type of support structure. Steel lattice towers and shell steel poles are commonly used.

The characteristic feature of shell poles is the form of centrally located load transferred to the foundation in the form of resultants of bending moment, torsional moment, horizontal force, and vertical force. Analogous impacts on the foundation are also transferred in the case of shell support structures in wind turbines. Detailed rules for defining loads and impacts for such structures, e.g. for small turbine sets with a rotor sweeping area up to 200 m^2 are given in the PN-EN 61400-2 (2008) standard (with updates). In general, for all structures of this type used in the energy industry, direct foundations in the form of various types of foundation slabs, and deep foundations in the form of slabs and pile caps, or monopiles are designed. This applies to the most common foundations for onshore applications, as the variety of foundation types for support structures in the energy sector outside the conventional land-sea boundary in the so-called offshore area is much greater. Due to their specificity, these structures require a different approach and are not considered in the present paper.

The focus of the paper is on selected practical aspects of designing a very effective type of foundation for power engineering structures in the form of monopiles, made using the well technique. They are most commonly used for making foundations of shell steel poles for overhead power lines in the voltage range from 45 kV to 400 kV.

The basis of the calculations for the monopiles structures are given in the well-known articles of Brinch-Hansen (1961) and Broms (1964). In Poland, the most important works on this subject were described by Dembicki (1971) and Dembicki & Odrobiński (1977). Currently monopile design is strongly developed for offshore wind structures Achmus (2009), Gupta (2020), Hao (2017), Ma (2020), Murphy (2018), Staubach (2020) and Yu (2017). Those are mostly typical concrete piles. Some works do concern piles with additional bearing plates Buslov (2015) or hybrid structures with steel tubes Bhowmik (2013) and Serras (2021). However, the authors could not find works describing the behavior of hybrid piles consisting of monolithic and precast concrete.

1. Monopile design in power generation

PN-EN 50341-1 (2013) and PN-EN 50341-2-22 (2016) standards specify the principles of designing shell steel poles that support overhead power lines. They also set out the requirements for the foundations of these structures. Unfortunately, neither the principles of their geotechnical nor structural dimensioning are specified in terms of the design of monopile foundations. All general recommendations related to foundations were referred to the basic standards PN-EN 1997-1-1 (2008) and PN-EN 1997-2 (2009). Structural requirements for reinforced concrete piles are governed by PN-EN 1992-1-1 (2008) while selected detailed issues of monopile forming and construction can be found in PN-EN 1536:A1 (2015). However, it should be noted that in general, PN-EN 1536:A1 (2015) standard applies to drilled piles intended for use in multi-pile support systems in bridge construction. Hence, among other things, it has a narrow range of application to piles with high slenderness, expressed as a ratio of their length to the diameter of over 5. This is related to the obvious need for sufficient slenderness in the pile group to redistribute the load to them. This is not applicable to monopiles, but formally the standard does not exclude this condition for them. However, in the design practice of the Polish energy industry, monopiles with slenderness below 5 are very often used. They were designed according to the outdated PN-B-03322 (1980) standard. In the standard, this restriction did not apply. In the PN-B-03322 (1980) standard, monopile foundations were classified as pile-column foundations and calculated according to an innovative procedure developed by the Institute of Hydroengineering of the Polish Academy of Sciences in Gdańsk. The methodology of the calculation was described in detail in the paper Dembicki (1971). The substantive basis for the developed method and many years of positive experience from its application in practice became the reason for approval of the computational models of foundations of supporting structures for electrical power lines, especially monopile foundations, for use currently based on the provisions of PN-EN 50341-2-22 (2016). This applies only to the computational model of the monopile, as the general rules used in geotechnics according to PN-EN 1997-1-1 (2008) remain obligatory.

The load capacity of a monopile (pile-column foundation) due to bending moment was calculated according to PN-B-03322 (1980) with the formula (1)

$$M_f = \nu_1 \nu_2 \overline{M} \gamma_D^r D^4 \tag{1}$$

where: $\nu_1 \nu_2$ – coefficients of soil cohesion and foundation shape, $\overline{M}$ – dimensionless value of the limit moment contained in the standard in tables (PN-B-03322, 1980) depending on the type of foundation, γ_D^r – calculated value of the volumetric weight of the soil around the foundation, D – foundation base depression (pile length).

It can be seen from formula (1) that the key factor determining the load capacity of a monopile according to the procedure contained in the standard PN-B-03322 (1980) is its depth, as it appears in formula (1) in the fourth power. However, there are many situations in practice when it is impossible to build a monopile of a specific length. If the load capacity deficiency is not great, increasing the pile diameter is an appropriate solution, although this is much less effective than increasing the pile length. The effect of the pile width (its diameter in case of circular cross-section, as from a formal standpoint, the standard PN-B-03322 (1980) also concerns monopiles with rectangular cross-section) is taken into account indirectly in the dimensionless value of the limit moment, which depends on the ratio of the pile depth to the radius of its cross-section (D/R).

For applications in the energy industry, mainly in the construction of support structures for overhead power lines, monopiles with diameters ranging from 1.2 to 3.0 m and lengths from 4.0 to 20 m are most commonly used. In practice, the construction of such structures is carried out by two different techniques adapted to the geotechnical and hydrological conditions and the loads transferred to the foundation (Fig. 1).

The first variety involves a drilling technique which uses large-diameter drilling rigs. It allows for the construction of piles of almost any length with a diameter of up to 1.8 m (so far in Poland) and up to 2.5 m abroad, which depends only on the availability of suitable drilling rigs. Instead of the expensive drilling technique, the well technique is used as an alternative. In this case, reinforced concrete rings are used as formwork. Piles with a diameter of up to 3 m can be constructed using the well technique. The disadvantage of this technique, however, is the depth limitation. Practical experience shows that it is effective (without loss of coaxiality of rings) to submerge typical rings to a depth of about 10 m. Therefore, in practice, it is assumed that for the required pile depth of up to 10 m, two construction techniques may be chosen, depending on local conditions, while for the depths of above 10 m only the drilling technique can be used.

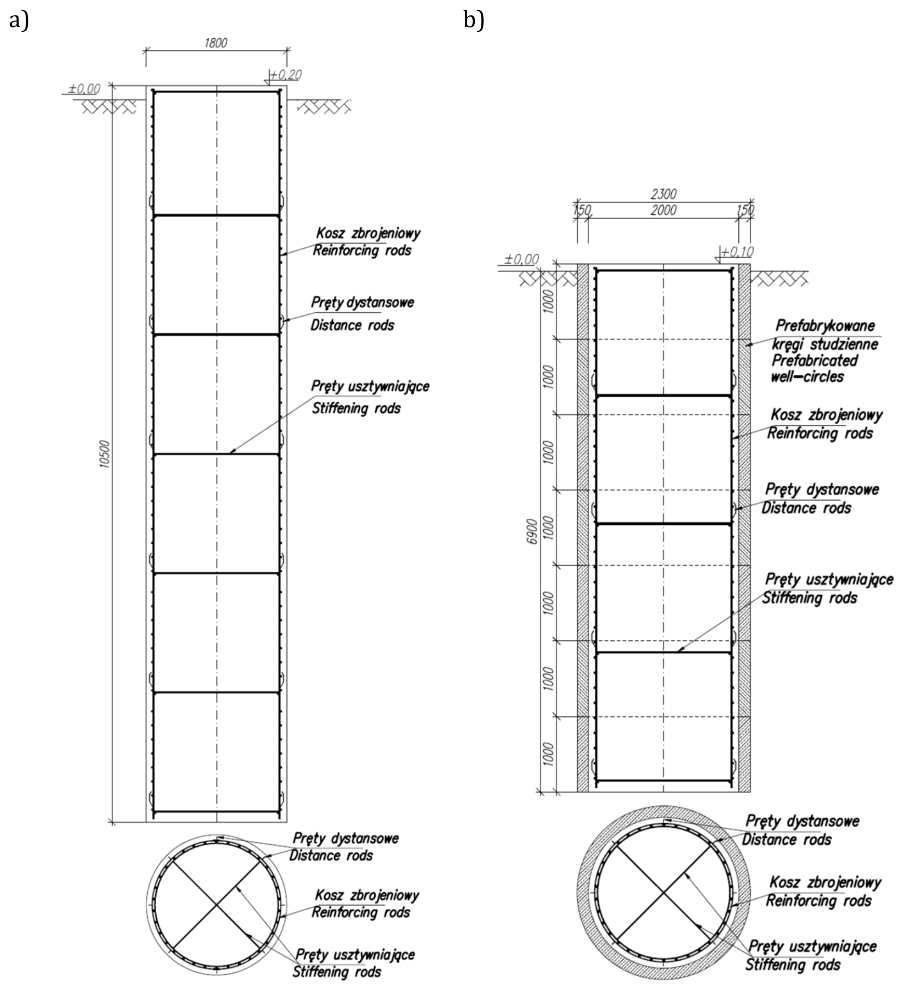

Fig. 1. Types of monopiles: a) driven pile, b) pile performed in the well technique (*own study*)

In the case of pile construction using the well technique, it is assumed that the load-bearing part of the pile structure is a reinforced concrete monolithic core made of rings, while the rings themselves represent a stay-in-place formwork.

The individual rings are not connected to each other, so that when bending the pile, separation of the individual rings in the form of a circumferential crack is possible and longitudinal normal stresses are transferred almost exclusively through the monolithic inner core. In practice, the calculation of the load capacity of the pile and the choice of its reinforcement are based on the internal cross-section. Furthermore, contact with the soil medium takes place on the external surface of the pile, i.e. as for structures with the external diameter of rings. Therefore, the load capacity of a pile in the ground can be determined by taking into account its outer diameter. In practice, designers use different approaches. One of them is the safe approach that assumes the outside diameter of the monolithic core in the evaluation of the load capacity in the ground. The other is the economical approach, assuming the outside diameter of the rings. In fact, due to resistive shear stresses and cohesion on the inner surface of the rings in contact with the core, they cannot move freely relative to it and the structure is partially composite. At the same time, the lack of cohesion between the rings in the horizontal planes of their contacts does not guarantee the full monolithicity of the structure. Obviously, the rings used also influence the decision on the approach used. According to the authors, it is a prerequisite to use only reinforced concrete rings. It should be noted that the rings up to a diameter of 1250 mm are standardized in PN-EN 1917 (2004), while for larger diameters, they are manufactured based only on individual manufacturer approvals. The inclusion of prefabricated rings in the evaluation of the load capacity of the foundation is only possible if they have sufficient load capacity and durability corresponding to the operating lifetime of the foundation.

To formally estimate the effects of the lack of full monolithicity of the structure and to evaluate the effect of the phenomena related to the separation of prefabricated elements on the load capacity of the soil, numerical calculations of the monopile were performed using the commercial software MIDAS GTS NX.

2. Numerical model of a monopile using the well technique

Several numerical models of the monopile studied were built in the MIDAS GTS NX software, which is based on the finite element method (FEM). The monopile shown in Figure 1b was modeled (structural symmetry was used) and it was founded in a stratified soil composed of sands, including water-saturated sands. In the basic version, the pile is modeled as a reinforced concrete model completely integrated into the rings. In the next version, the pile was modeled with rings that had a separation option at the contact with the core through non-linear Coulomb-type contact and cohesion at 10 kPa. The soil medium was modeled as an elastic-plastic material meeting the criteria of Coulomb-Mohr plasticization and interference with linear-elastic concrete foundation, also based on Coulomb-Mohr theory but with a reduction factor of 0.7. The geotechnical parameters of the stratified ground in the model are shown in Figure 2. Furthermore, a general view of the meshed FEM model is shown in Figure 3. Non-linear elastic analysis of the structure was carried out by loading the model in the foundation head with the resultant design loads of a bending moment of 2729.6 kNm, horizontal force of 96.2 kN, and the vertical compressive force of 518.8 kN.

A comparison of the deformation of the monolithic structure and the prefabricated structure is shown in Figures 4 and 5. The deformation pattern of both models is very similar. The ratio of maximum displacement values of both models is 0.04094/0.03841 = 1.066 i.e. the global maximum displacement of the prefabricated structure is about 7% higher than that of the monolithic structure. Furthermore, the relative deformations of the individual rings are observed in the prefabricated structure model. The values of stresses assumed in the prefabricated model at the contact between the rings and the core may be actually higher and then the difference of real displacements of the structure can be even smaller.

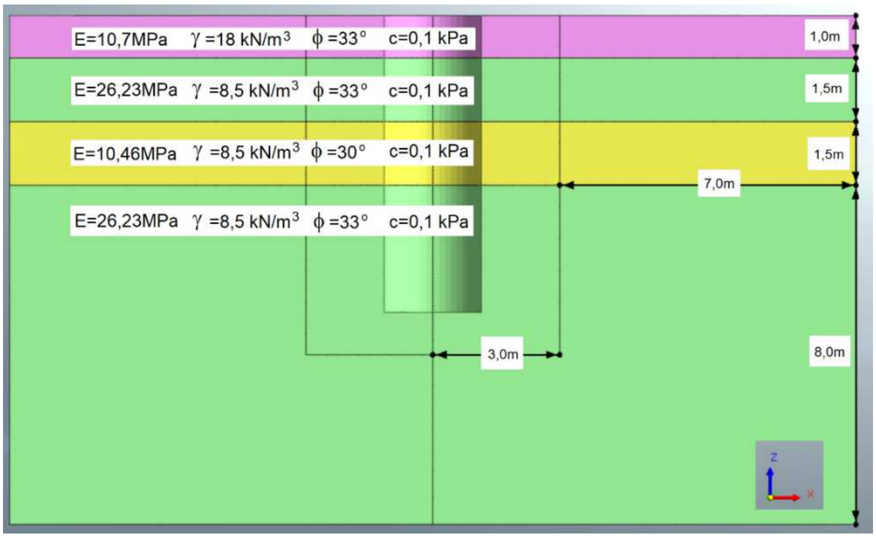

Fig. 2. Soil layers and geotechnical properties of soil used in the analysis (*own study*)

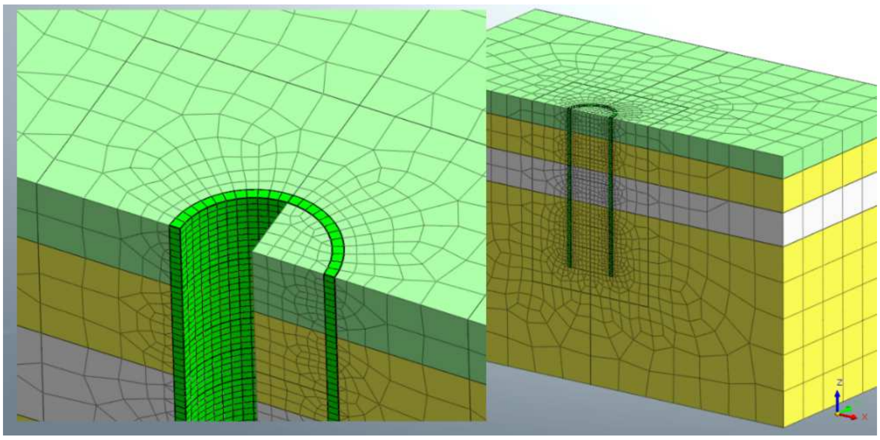

Fig. 3. View of the geometry and FE mesh (*own study*)

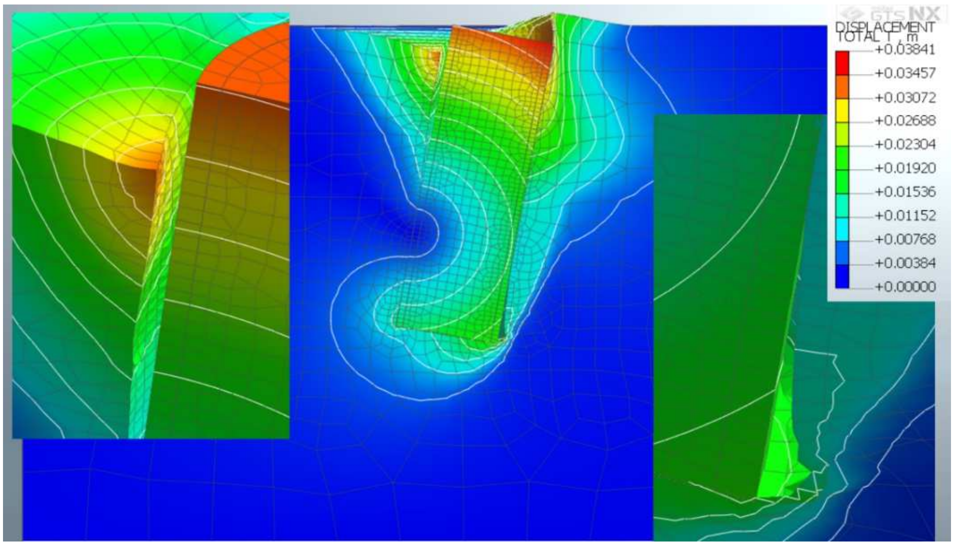

Fig. 4. Deformation of the monolithic structure (*own study*)

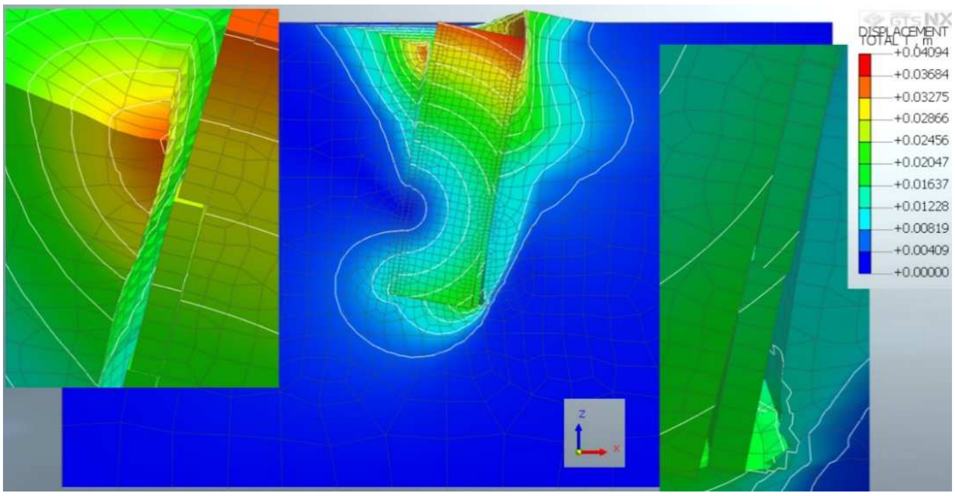

Fig. 5. Deformation of the precast structure (*own study*)

It should be noted, however, that the described pile working mechanism depends on the load capacity of the prefabricated rings themselves. Figure 6 shows an example of normal stress maps in rings (S-YY in kPa, horizontal Y-axis direction) for a monolithic pile model and a pile model with prefabricated rings. As can be seen, the tensile stresses in the monopile with prefabricated rings more than double from 406 to 997 kPa, reaching the tensile strength level for weaker concretes. Obviously, due to model simplification, these stresses should be approached as estimates. Nevertheless, the results obtained clearly indicate the importance of the problem and show that the effect of the rings on the structure's load capacity cannot be uncritically assumed without their strength evaluation.

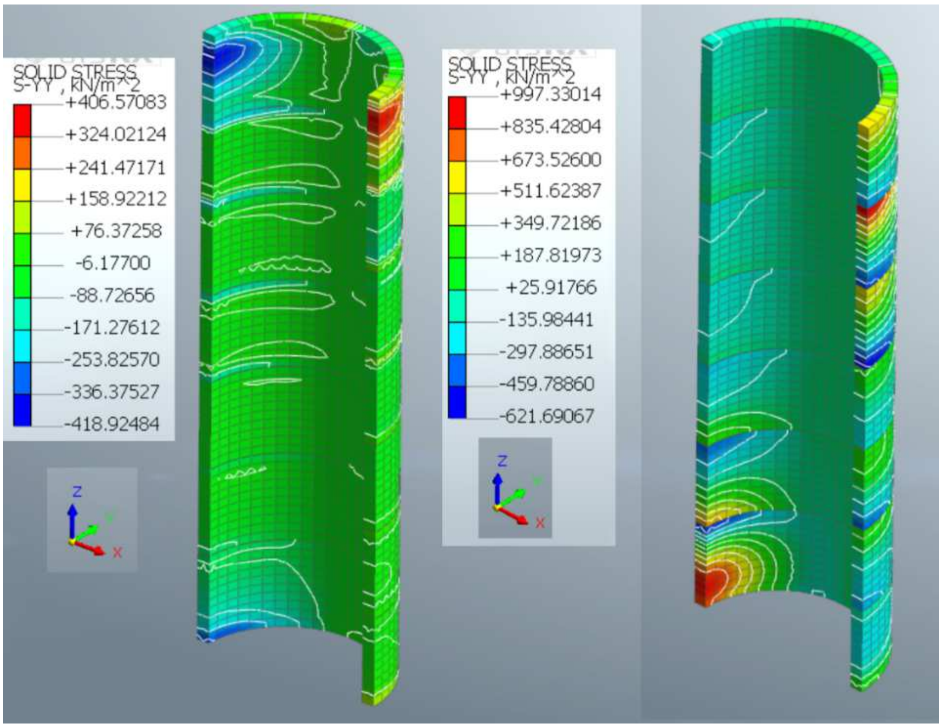

Fig. 6. Maps of normal stresses in a monolithic pile (left) and a precast pile (right) (*own study*)

In the final stage of calculations, physical limit capacities were calculated for both reference models. A nonlinear static analysis was carried out for the models by carrying out calculations for the increasing bending moment until the mechanism was obtained. It was assumed that in the case studied, this occurs when the convergence of the non-linear analysis process is lost. Figures 7 and 8 show the static equilibrium paths defined for a point in the axis of the pile tip depending on the bending moment versus its horizontal displacement up to the limit of 500 mm. These figures also show tangents (with their equations) to the initial and final sections of the static equilibrium curves, with their intersection, according to the method described in PN-EN 61773 (2000), determining the limit load capacity of the foundation. The limit load capacity of the monolithic foundation determined in this way was 5037 kNm (Fig. 7), while the limit load capacity of the foundation with prefabricated rings was 5009 kNm (Fig. 8), i.e. it was lower by only about 1%. Assuming the global factor of safety in accordance with PN-EN 1997-1 (2008) of 1.4, the designed load capacity of the analyzed foundation is 5009/1.4 = 3578 kNm > 2729.6 kNm (the stress utility for the limit load capacity state for the design moment is approximately 76%).

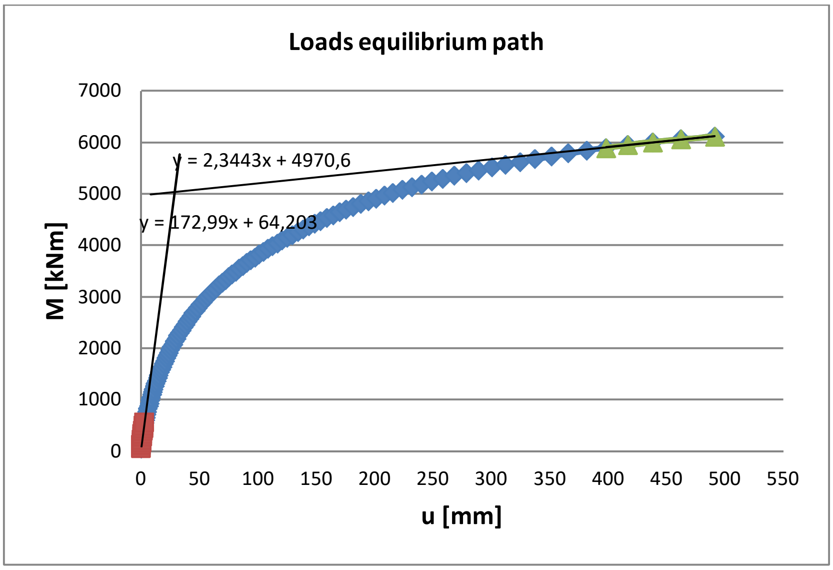

Fig. 7. Loads equilibrium path for a monolithic pile (*own study*)

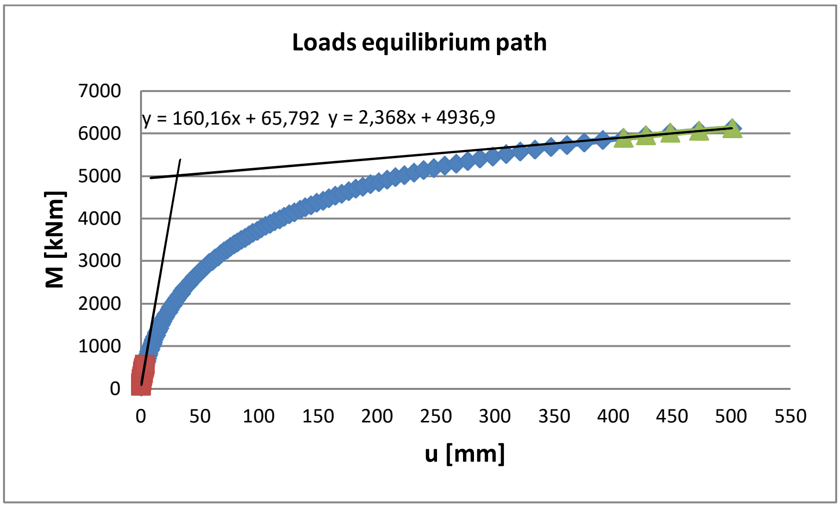

Fig. 8. Loads equilibrium path for a precast pile (*own study*)

Conclusions

The construction of monopiles intended for applications in the power engineering industry using reinforced concrete rings is effective and economical, although practical experience in the field shows that this technique is particularly applicable only to pile lengths up to approximately 10 m. In the opinion of the authors of the present study, when designing this type of monopiles for the assessment of their geotechnical load capacity in the soil, their external diameter (i.e. including rings) can be taken into account. As can be seen from the presented example, the non-linear effects related to the lack of full bonding between the prefabricated rings and the monolithic core do not cause adverse effects that reduce the functional parameters of the monopiles designed in this way by more than about 7%. Despite the authors' positive practical experience with this type of structure, quantitative conclusions in this regard cannot yet be generalized. Furthermore, when accepting such solutions, it must be taken into account that the rings from the conventional function of the stay-in-place formwork become an element of the foundation structure and must therefore also meet the relevant load capacity conditions of their strength.

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