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DOCTORAL THESIS

- *SUMMARY* -

Modern approaches for advanced calculation of port foundations on piles under cyclic stresses

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MODERN APPROACHES FOR ADVANCED CALCULATION OF PORT FOUNDATIONS ON PILES UNDER CYCLIC STRESSES

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Foundation soil, floating piles, cyclic stresses, port structures, subsidence, foundation, bearing capacity, indirect foundation, load-discharge, compression-return, testing, deformation, shear, peak pressure, influence factor, lateral friction, drilling.

3. INTRODUCTION TO THE THESIS TOPIC

Due to the inhomogeneous and poorly load-bearing nature of the foundations and the intensity and variability of the actions induced by the superstructure of these constructions, the infrastructures of industrial port objectives often require the use of indirect foundations using floating piles. These pile foundation solutions are frequently used in civil engineering in the context of the lack of foundation soil with sufficient mechanical characteristics in the lithological stratification of the studied site to be able to absorb the loads generated by the superstructure by means of direct foundation.

Most of the external loads are axial compressive loads, but the piles are also capable of taking axial tensile loads as well as lateral forces (from wind, waves, earthquakes, etc.).

A special category of structures relating to port operations is represented by buildings in which bulk goods, especially grain, are handled and which comprise high storage buildings in the form of metal silos.

These types of structures are usually founded on piles because of the high loads to be transferred to the ground, and also because of the existence of weak layers on site in terms of bearing capacity (e.g. the nature of the fills used in the process of widening/forming harbour berths).

Typically, grain cells have a capacity of between 2,000 tonnes – 15,000 tonnes, resulting in a diameter of 15.0-25.0 m and running heights of up to 30.0 m. The new grain systems (cells) are built on a metal structure, using thin-walled plates and profiles made of high-quality steels (S355). These storage systems are more efficient in terms of cost-effectiveness by using prefabricated solutions that are light and quick to implement.

However, there are also specific features of these constructions, which require a specific approach to infrastructure design solutions, in particular due to the cyclical nature of loading and unloading and the high ratio of live loads to permanent loads:

- Limitation of differential heave values to avoid tilting of the structure (and therefore ensiling capacity, limitations of machine operation, etc.), which is a maximum of $2‰_0$. For a grain cell diameter of 27.00m, compliance with this limit implies limiting the differential heave value to a maximum of 5.40cm.
- The major difference between the dead weight of the structure and the variable load (bulk grain). In the case of a 10,000-tonne capacity silo, the deadweight of the structure is approximately 150 tonnes (representing approx. 1.5% of the normalised live load in

the basic calculation grouping). This aspect, in conjunction with the cyclical nature of the loading/unloading of the goods, leads to a different approach to determining the bearing capacity of a pile within the indirect foundation system of the steel silo infrastructure.

- Grain silos are loaded and unloaded cyclically over a relatively small interval (a few days) so that the loading on the piles also varies cyclically from highs to lows. It is necessary to ensure that the deformations that occur over time do not lead to malfunctions in the operation of belt or scraper conveyors, tubing, bunkers, stakes, etc...

The highly demanding nature of the cyclic actions mentioned above (low frequency, taking into account the existence of an average number of 10-12 annual load-discharge cycles) makes their consideration complex. Variable actions are generally predominant in these types of structures, characterised by high values relative to dead loads, and cyclic parameters are often ignored in the design of the infrastructure system. This type of loading is probably the most complex mode of foundation soil loading, and modelling such loading requires analysis of both the soil-structure interaction and the cyclic loading-unloading characteristics of the soil.

At present, the normative basis for design in the field of indirect pile foundation requires dimensioning methods, in-situ or small-scale experimental tests and calculation programs for a vertically statically loaded pile foundation with consideration of soil-structure interaction (e.g. *SR EN 1997-1:2004* or *NP 123-2010* - "Standard for the geotechnical design of pile foundations"). Thus, from this point of view, the national and international information base on pile foundations required vertically static is well founded, there are explicit norms and standards in this regard, and a multitude of scientific works have been carried out over time on this subject.

However, given the highly particular character of these bulk storage facilities (in the case study - grain) by using metal cells made of thin-walled sheets and profiles (working with active compression zones) in conjunction with a poorly load-bearing infrastructure, the identification of structural design solutions by considering the dynamic character of long-term operation becomes very complex and complicated.

It is important to note that Eurocode 7 (*SR EN 1997-1:2004*) recommends that the effect of cycles should not be neglected but it does not provide any solution in this respect. Also, in SR EN 1997-1, there is no reference to the use of prescriptive methods for axially stressed piles (see also subchapter. 7.5.1.Piles exposed to axial loads - Compressive bearing capacity in GP 129/2014 - Geotechnical design guidance). In this sense, the use of these methods should only be of an indicative and pre-dimensioning nature in the design of pile foundations.

At the same time, it is noted the lack, in the regulatory base in force, of a complex and exhaustive numerical modelling on the subject in question, including a parametric approach to identify the ultimate bearing capacities of these types of piles vertically cyclically loaded at low frequencies.

Therefore, the present PhD thesis, whose subject is of interest for objectives located in the port field and not only, had as main objective to study the problem of piles vertically cyclically stressed (low frequency) from cyclic actions such as loading and unloading of grain in silos. The approach involved both a static (prescriptive) analytical dimensioning calculation carried out according to NP123-2010 and NAVFAC DM 7.2 (GEO5-Piles calculation program) and a detailed analysis of the test results of sample piles from the site of berth 80 Port Constanta, loaded at a number of 8 (eight) loading-unloading cycles.

The general and specific objectives set for the PhD thesis were as follows:

- To produce a bibliographical synthesis, with support from the international literature, on the behaviour of foundations on piles under cyclic vertical load;
- Comparison of the numerical results of the estimated ultimate bearing capacities of the piles, based on the prescriptive calculation methods in force, with the results of actual field tests of a vertically cyclically loaded pile.
- Recommendations for the design of foundations on vertically cyclically (low frequency) stressed piles, based both on the interpretation of test results on test piles at the study site and from the experience of previous similar projects carried out in the Port of Constanta.

4. SUMMARY OF THE CONTENT OF THE DOCTORAL THESIS

The PhD thesis was structured in five main parts as follows:

4.1. PART I - BIBLIOGRAPHIC AND LITERATURE REVIEW ON THE BEHAVIOUR OF FOUNDATIONS ON PILES

In this part, materials from the literature both in the field of geotechnics (earth mechanics) and in the field of pile foundation design (standards in force, calculation methods, settlement evaluation, etc.) were studied.

The study of the compressibility of soils is complex since each lithological layer in the structure of the land practically supports the load coming from the layers above it, also called *geological load* or geological pressure, and noted with σ.

Assuming that the ground is a continuous medium, a mathematical relationship can be established between the pressure exerted and the resulting deformation. Thus, the value of the deformation mode of the structure of the whole earth mass considered is significantly lower than the value of the deformation mode of the granules making up the mineral skeleton of the earth. This difference led to the hypothesis that the deformation of the granules of the solid skeleton was neglected compared to that of the basic earth, the granule assemblage and that the deformation of the earth was mostly due to the decrease in its porosity.

Reducing the porosity of the soil leads to a (denser) resettling of the granules, and an increase in the number of contact surfaces as well as in the pressure exerted on them. This leads to a reduction in the thickness of the adsorbed water films, thus reducing the distance between the centres of gravity of the particles. As a result, there is an increase in the surface area of the contact zones between the particles, which results in an increase in the earth's resistance to external loads.

Thus, it is necessary to analyse not only the phenomenon of charging the earth but also the phenomenon of charging, discharging and recharging it (**Fig 2.3**.).

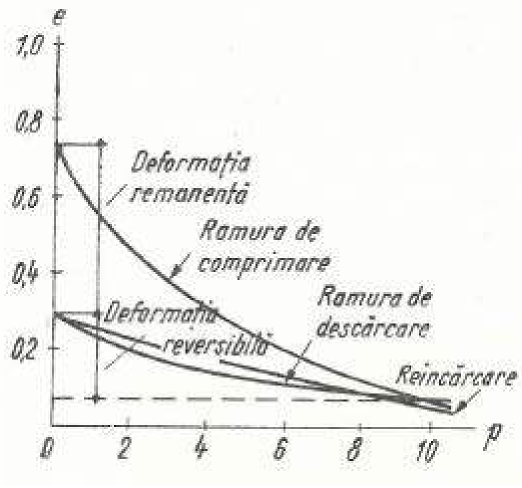


Fig. 2.3. Compression-load curve: loading-unloading-reloading [84]

In practice, short and repeated stresses (high-frequency cyclic stresses) are also encountered, especially in machine foundations. Thus, the deformation of the earth is achieved in a relatively short time as a result of an impact stress state, there being insufficient time for a full deformation similar to a single load situation to materialise.

Under equal pressures, it is found that the cyclicity of the stresses (load-unload cycles) produced successively over time results in an increase in the value of the total deformation of the bedrock. At the same time, according to the graph in **Fig. 2.4**, it can be seen that the deformations corresponding to each load decrease.

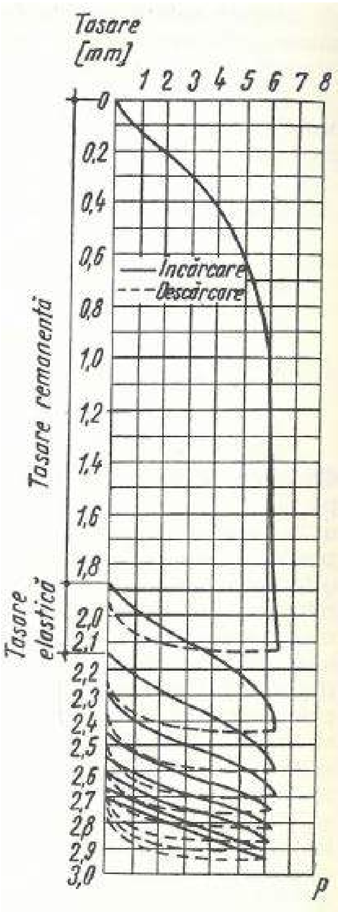


Fig. 2.4. Compression-load curve under repeated loading[84]

It was also observed that the remanent tails decrease faster than the elastic ones with an increasing number of load-unload cycles. Thus, after a large number of load-unload cycles, the foundation soil behaves "elastically" in terms of the values of the newly materialised deformations. It can be seen that the deformations are almost elastic in the sense that the curves of loading coincide with those of unloading.

In practice, however, the different appearance of the two load-discharge branches is due to the fact that at normal deformation rates, the deformation cannot fully materialise, giving rise to a hysteresis loop.

An ideal calculation model for determining the load-bearing capacity of the single pile would be one that allows the specialist to identify the true load-load relationship to failure (as per Terzaghi's hypothesis), for any type of pile and for any site conditions. This, however, is very difficult to implement in practice for the current state of knowledge on the calculation of foundations on (floating) piles. The final determination of the ultimate compressive bearing capacity of the pile should be based on a more rigorous analysis of the theoretical and experimental results, also taking into account the influence of site-specific factors (terrain, adjacent constructions, etc.).

In order to establish the computational relations by means of semi-theoretical models, a number of simplifying assumptions have to be considered from the beginning as follows:

- The pile is represented by a homogeneous, non-deformable cylindrical body;
- The cross-section of the pile is constant vertically;
- The pile base is laid out in a homogeneous terrain;
- A vertical, axial, compressive load acts on the pile;
- Along the length of the pile, the terrain intersected by the pile may be homogeneous or composed of several different layers.

In this respect, the ultimate strength of the indirect foundation system is determined by:

- determining the resistance of the ground to destruction below the pile tip;
- determination of the slip resistance of the ground adjacent to the lateral surface of the pile's body.

Slip resistance is determined as the sum of two types of resistance:

- to slide past the side surface of the pile as well;
- to the loss of stability of the dislodged earth under the pile tip.

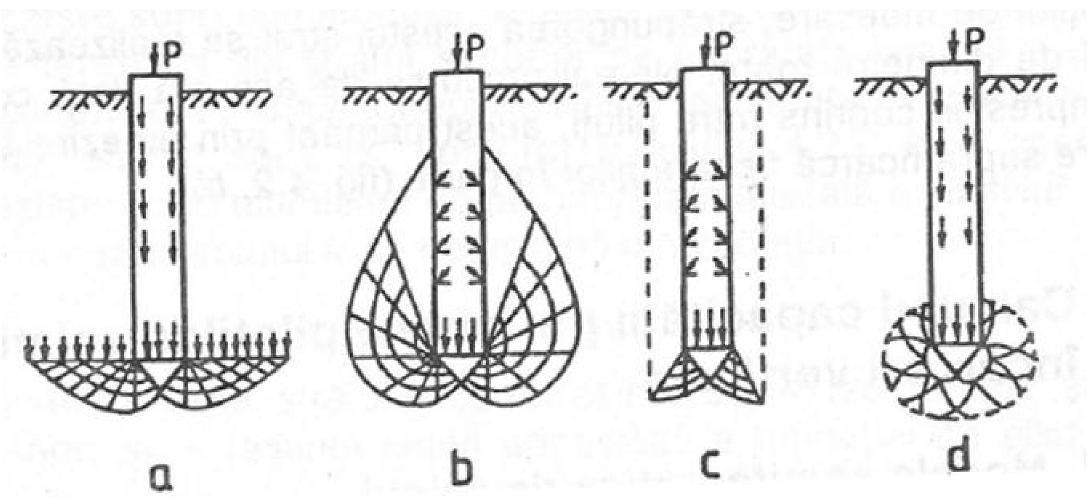


Fig. Methods of limit equilibrium under deep foundations [60]**:**
a - after Prandtl, Reisner, C. Kerisel, Terzaghi, Buisman; *b* - after De Beer, Meyerhof, Djaki; *c* - after Berezanțev, Irașenko, Vesic; *d* - after Bishop, Hill, Matt, Skempton, Yassin and Gibson.

The identification of a computational relationship can be performed based on the theory of earth limit equilibrium, which determines the qualitative aspect of destruction of the earth that makes up the foundation soil of deep foundations. It is known that the characteristic shape of the destruction zone (of the limit equilibrium), determined by using the indirect foundation system (piles), is different from that created by using surface foundations. In **Fig.3.1.** some theoretical models of the limit state zones for pile foundations, established by a number of renowned researchers in this field, are presented.

NORMATIVE (REGULATED) CALCULATION METHODS

EUROCOD 7 [SR EN 1997-1]

Regarding the calculation of piles, EC7 allows the design phase to be based on the following considerations (principles):

- analytical calculations or calculations based on empirical determinations are validated by static tests on test piles;
- static tests on test piles are relevant in the sense of relating to situations of actions and behaviour similar to the real model;
- dynamic tests on test piles are validated by reference to static tests carried out in comparable situations. Dynamic tests refer to methods using time records of the forces applied to the pile's head and the pile's displacement response during dynamic impact (<1sec).
- consideration of comparing test results on different piles (in similar ground-effort situations) to validate the ground response in the test to determine the ultimate bearing capacity of the pile.

The general equation, valid for all load cases and load groups related to SLU, for validating the compressive bearing capacity of an indirect foundation is:

$F_{c,d} \leq R_{c,d}$,where:

$F_{c,d}$ = the calculation value of the axial compressive load on a pile or group of piles;

$R_{c,d}$ = design value of the compressive strength of the ground under the pile at the ultimate limit state.

Thus, according to EC7, the determination of the ultimate design compressive bearing capacity ($R_{c,d}$) will be based on:

- analysis of the test results of the test piles that will be executed in the same way as the piles that make up the foundation and must be founded in the same layer (see chapter 7.6.2.2. EC7);
- analysis of soil test results (see chapter 7.6.2.3. EC7);
- analysis of dynamic impact test results (see chapter 7.6.2.4. EC7);

- the beating formulae, which will only be used if the terrain stratification is known and if the minimum requirements for method validation are met (minimum 5 piles tested, etc. - see chapter 7.6.2.5. EC7);
- interpretation of the wave equation only if the ground stratification has been determined by boreholes and field tests (see chapter 7.6.2.6. EC7)

NP123-2010

The provisions of NP120-2010 on the geotechnical design of pile foundations are related to the principles set out in section 7 of EC7, but in addition to this, it introduces prescriptive methods for determining the ultimate bearing capacity of floating and top bearing piles (as set out in chapter 7.2.4. of NP123-2010).

Thus, by referring to certain predefined values of the characteristic values of the pile base pressure and lateral frictional resistance in relation to the characteristics of the earth, the characteristic value of the compressive bearing capacity of a pile can be determined. Subsequently, depending on the partial safety coefficients taking into account the design of the pile, the nature of the soil at the base and the length of the pile, the design value of the compressive strength of a pile can be determined.

The prescriptive method, even if not referenced in EC7, allows a fast and efficient pre-dimensioning of the design value of the compressive strength of a pile, given a good knowledge of the soil stratification and geotechnical parameters of the earth, determined by representative boreholes, field tests and laboratory work.

4.2. PART II - PERSONAL CONTRIBUTIONS ON THE THESIS TOPIC

This part was devoted to static calculations on the determination of the ultimate compressive bearing capacity of a reinforced concrete displacement pile (floating type) by various prescriptive methods and to the analysis of the test results of the test piles at the site of the 80 Constanta harbour dock, subjected to cyclic load-unload tests through 8 (eight) cycles.

The test piles are part of the infrastructure project of a grain terminal with a capacity of approx. 198,900 tons, consisting of:

- a group of 4 cells with a capacity of 10,000 tonnes;
- a group of 8 cells with a capacity of 10,000 tonnes;
- a group of 6 cells with a capacity of 10,900 tonnes;
- a group of 6 cells with a capacity of 2,250 tonnes;

and located on the port territory behind berth 80 in the Port of Constanta South (see Fig.5.1). The infrastructure for the grain cells, both for the 10000/10900-ton capacity and for the 2250-ton capacity cells, is of indirect type: 100cm thick general landfill arranged on a network of 30 respectively 10 piles of 900mm diameter, according to the layout of the formwork/execution plans (see Fig.5.2).

For the pre-dimensioning stage of the design values of the frictional resistance on the lateral surface of the pile, the resistance on the base as well as the ultimate compressive bearing capacity, prescriptive design methods were used as detailed in NP123/2010 and NAVFAC DM 7.2.

Also, using the results of the full-scale working model (test pile), a comparative analysis of the influence of loading cyclicity in determining the final value of the ultimate vertical load-carrying capacity of a pile at the studied site was performed.

FIG.5.1_Overview - Grain terminal at berth80 - port of Constanta

An important condition for the safe operation of the grain terminal's service technology was the limitation of differential subsidence between the buildings supporting its specific equipment. In this respect, the differential subsidence that may occur between the pile groups of the grain silo infrastructure and the metal towers supporting the cargo elevators was also taken into account.

Structures that transmit important vertical loads to the foundation ground are represented by grain grain cells. The specific characteristic of grain terminals in port areas is the short-term storage of bulk cargo and the high cyclicity of cargo transit during a year (minimum 10-12 loading/unloading cycles). This aspect, which is also conditioned by the imposition of short vessel operating times at the port berth, requires the implementation of cyclical actions such as loading and unloading of grain silos. Thus, cyclic loading and unloading stresses on reinforced concrete piles can condition their load-bearing capacity through the cumulative value of the remaining settlement associated with each cycle.

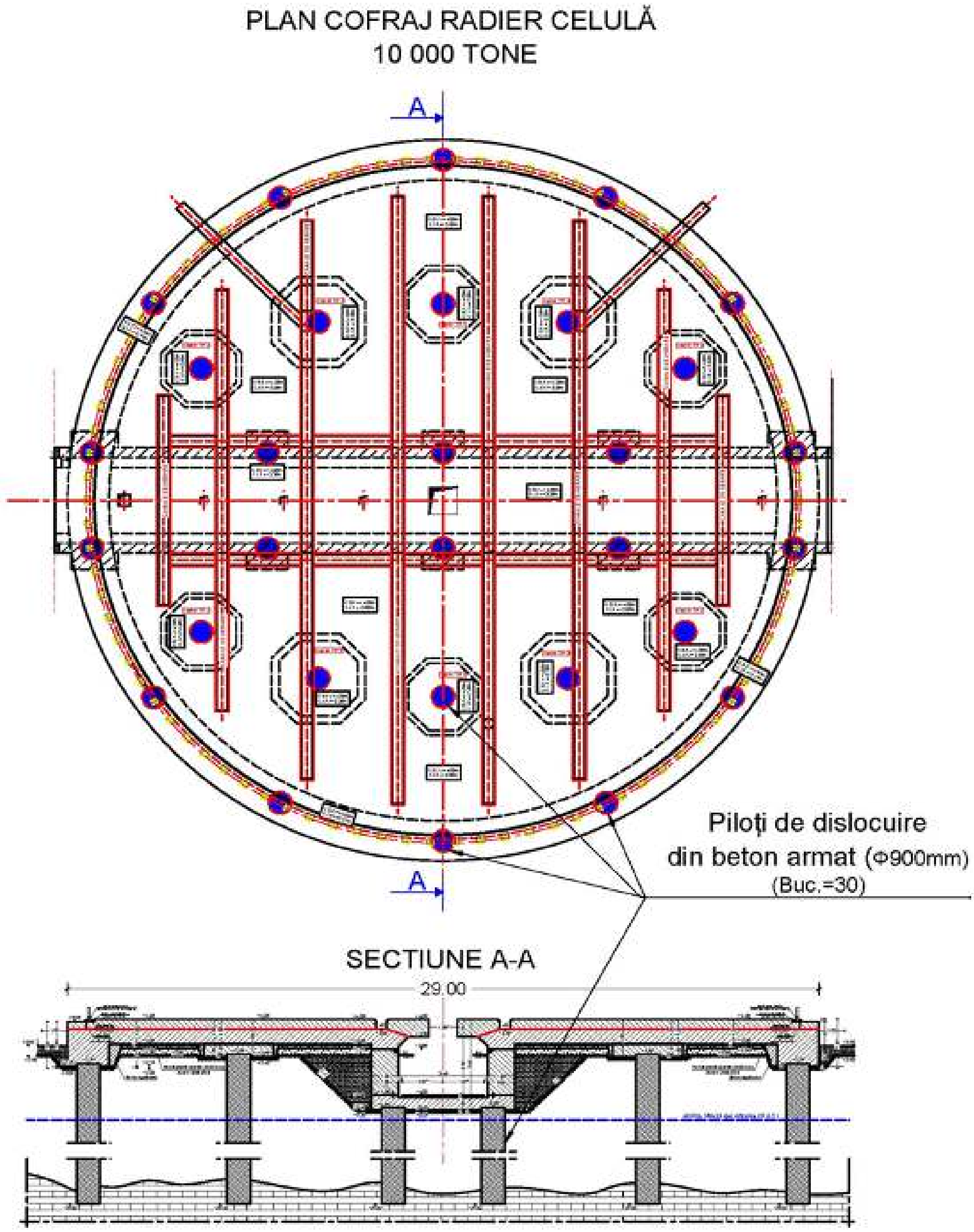


Fig.5.1._Formwork plan for the grain cellar - capacity 10,000 tons

The geotechnical investigation for the construction located behind the 80 berth, Port Constanta South was done by drilling six geotechnical boreholes of 40 m deep. In order to complete the information on the last intercepted layer, three of the drilled holes were continued to a depth of 42.00 m.

In order to simplify the sequence of the intercepted strata, they have been grouped into 3 lithological complexes, resulting in the following synthetic lithological column of the site:

- C1 - the complex formed by anthropogenic fill (predominantly composed of cohesive strata): clay, dusty clay, sandy clay, clayey dust; weakly cohesive strata: clayey sand, dusty sand, clayey sandy dust and non-cohesive strata: fine sand - large, with small gravel. All this succession of strata contains boulders and limestone blocks. At the top of drill holes F1, F2, F3 and F6, iron ore deposits from the deposition of ore over time have been traced over a variable thickness of 0.10 to 0.80 m;
- C2 - intermediate complex composed of an alternation of: cohesive strata (clay, greasy clay, dusty clay, sandy clay, clayey dust) weak cohesive strata (clayey sand, dusty sand, sandy clay dust) and non-cohesive strata (fine sand, fine - large sand, large - fine sand, large sand, small-large gravel, faults. All this sequence of strata contains concretions and limestone boulder elements);
- C3 - lower complex - C3 is composed of an alternation of chalky clay / chalky, whitish dusty clay with concretions and limestone boulders. Drilling was stopped in this complex without crossing it (at depths of 40.00 - 42.00 m).

The table below shows the depth intervals over which the 3 lithological complexes develop as well as their thicknesses, according to each geotechnical drilling:

Denumire foraj	Adâncime foraj (m)	Complexul 1 (umplutură: strate coezive în alternanţă cu strate slab-coezive şi necoezive, cu concreţiuni, pietriş, bolovniş şi blocuri de calcar)		Complexul 2 (alternanţă de roci nisipoase/argiloase cu concreţiuni şi bolovaniş de calcar)		Complexul 3 (*) (argila cretoasă/argilă prăfoasă cretoasă cu bolovăniş calcaros)	
		Interval adâncimi (m)	Grosimi (m)	Interval adâncimi (m)	Grosimi (m)	Interval adâncimi (m)	Grosimi (m)
F1	42.00	0.00-9.40	9.40	9.40-30.80	21.40	30.80-42.00	11.20
F2	40.00	0.00-16.30	16.30	16.30-32.20	15.90	32.20-40.00	7.80
F3	42.00	0.00-16.00	16.00	16.00-35.20	19.20	35.20-42.00	6.80
F4	40.00	0.00-12.00	12.00	12.00-30.20	18.20	30.20-40.00	9.80
F5	40.00	0.00-11.30	11.30	11.30-31.60	20.30	31.60-40.00	8.40
F6	42.00	0.00-11.80	11.80	11.80-35.80	24.00	35.80-42.00	6.20

(*) This complex was not traversed by the drill holes, the thickness only represents the penetration depth of the drill hole in the start. Groundwater was intercepted in all 6 boreholes, generally in the depth range 2.37 ÷ 4.50 m.

DETERMINATION OF THE COMPRESSIVE BEARING CAPACITY OF THE PILE BY PRESCRIPTIVE METHODS (CHAPTER 7.2.4.2 - FLOATING PILES), ACCORDING TO THE PROVISIONS OF NP123/2010.

Pile classification:

- By material: - **reinforced concrete**;
- According to the effect that the pile deployment process has on the surrounding terrain: - **displacement**;
- By variation of cross-section: - **with constant cross-section**;
- According to the way of execution: - **execute on-site by drilling;**
- By diameter size: - **large diameter (900 mm)**;
- Depending on the way the hole walls are supported, the piles are drilled in place: **- drilled with recoverable casing;**
- According to the direction of stress relative to the longitudinal axis: - **axial and transverse stresses are applied simultaneously;**
- According to the mode of transmission of axial loads to the ground: **- floating**;
- According to the position of the longitudinal axis: **- vertical**.

Notations used:

- Rc = compressive strength of the ground in contact with the pile at the ultimate limit state;
- Rc.d = the calculation value of Rc;
- Rb.d = design value of the pile base resistance;
- Rb.k = characteristic value of the pile base resistance;
- gb = partial coefficient for the base resistance of the pile;
- Rs.d = design value of the frictional resistance on the side surface of the pile;
- gs = partial coefficient for frictional resistance on the side surface of the pile;
- Ab = area of the pile base;
- qb.k = characteristic value of the pressure on the base;

Geometric features:

- pile diameter: d=900 mm;
- the pile's actual sheet (measured from the natural ground level, the upper elevation of the dock platform): $D = 27{,}50\ m$

Lithological layers:

Layer 1: fill (clayey dust, sandy clay, dusty clay, plastic consistent...hard, with limestone boulders, with limestone blocks, with hydrocarbon smell, with grey areas).
 - thickness of layer 1 = 11.6m, Ic considered = 0.7;

Layer 2: small gravel and large sand, greyish, with shell fragments, weak odour hydrocarbons.

Layer 3: dusty, greyish, plastic, swirly clay with rare shell fragments, with black areas (Ic=0.8).

Layer 4: grey, consistent plastic clay with rare shell fragments, with blackish areas (Ic=0.7).

Layer 5: fine-grained sand with small, greyish gravel, unevenly grained (top) and evenly grained (bottom), with shell fragments, with limestone boulder.

Layer 6: yellowish-grey hard clay with FeO, with limestone boulder (Ic=1.0).
Layer 7: yellowish-white, clay sand, peaty plastic (at the top) and consistent plastic (at the base), with limestone boulder.
Layer 8: sandy clay, whitish-yellowish, plastic swirly, with boulders limestone.
Layer 9: small gravel and large, whitish, unevenly-grained sand.

Determination of the calculation value of the ultimate bearing capacity (Rc.d).

The partial safety coefficients are considered according to Tables 7 and 8 of NP123/2010, taking into account the pile design technology as well as the nature of the foundation ground, as follows**:**
- Pile concreting technology: underwater concreting;
- Type of soil at the base of the pile: non-cohesive;
- Pile execution: drilled with recoverable casing;
- Type of soil around the pile: cohesive.

$\gamma_{b.2} := 1.3 \qquad \gamma_{s.2} := 1.9$

Thus, the design value of the ultimate compressive bearing capacity of floating piles, executed in situ, is**:**

$$R_{b.d} := \frac{R_{b.k}}{\gamma_{b.2}} \qquad R_{b.d} = \left(5.604 \cdot 10^3\right)\ kN$$

$$R_{s.d} := \frac{R_{s.k}}{\gamma_{s.2}} \qquad R_{s.d} = \left(2.523 \cdot 10^3\right)\ kN$$

$$R_{c.d} := R_{b.d} + R_{s.d} \qquad R_{c.d} = \left(8.127 \cdot 10^3\right)\ kN$$

DETERMINATION OF PILE COMPRESSIVE BEARING CAPACITY BY PRESCRIPTIVE METHODS, USING GEO5 PROGRAM - PILES, ACCORDING TO NAVFAC Design Manual 7.2.

The calculation presented in detail in **Appendix no.1** of the PhD thesis (Analysis of the bearing capacity of the 900mm pile with actual slab D=27.50m), established a calculation value of the bearing capacity for the pile considered **Rcd=6224 kN** (see table below).

At the same time, in order to narrow down the range of values for the determination of the actual pile sheet, a detailed analysis of the bearing capacities between depths 10m-36m (in 1m increments) was carried out as shown **in Annex 2** (Analysis of the bearing capacity of 900mm piles with variable actual sheet D=10m-36m).

ANALYSIS OF THE VARIATION OF CARRYING CAPACITY IN RELATION TO THE ACTUAL PILE BEAM (10m - 36m) VALUES ACCORDING TO ANNEX 2										
Crt.	D (pile's actual file)	Pile diameter - ϕ	**Rsd**	γ_s2	Rsk	**Rbd**	γ_b2	Rbk	**Rcd**
	[m]	[mm]	[kN]	Tab.8	[kN]	[kN]	Tab.7	[kN]	**[kN]**
1	10	900	408	1,9	775	161	1,3	209	**569**
2	11	900	463	1,9	880	161	1,3	209	**624**
3	12	900	548	1,9	1041	119	1,3	155	**667**
4	13	900	589	1,9	1119	119	1,3	155	**708**
5	14	900	628	1,9	1193	119	1,3	155	**747**
6	15	900	668	1,9	1269	119	1,3	155	**787**
7	16	900	724	1,9	1376	476	1,3	619	**1200**
8	17	900	792	1,9	1505	143	1,3	186	**935**
9	18	900	841	1,9	1598	143	1,3	186	**984**
10	19	900	889	1,9	1689	143	1,3	186	**1032**
11	20	900	930	1,9	1767	143	1,3	186	**1073**
12	21	900	968	1,9	1839	113	1,3	147	**1081**
13	22	900	1006	1,9	1911	113	1,3	147	**1119**
14	23	900	1044	1,9	1984	113	1,3	147	**1157**
15	24	900	1079	1,9	2050	94	1,3	122	**1173**
16	25	900	1170	1,9	2223	4184	1,3	5439	**5354**
17	26	900	1326	1,9	2519	4376	1,3	5689	**5702**
18	27	900	1482	1,9	2816	4568	1,3	5938	**6050**
19	27,5	900	1560	1,9	2964	4664	1,3	6063	**6224**
20	28	900	1638	1,9	3112	4760	1,3	6188	**6398**
21	29	900	1695	1,9	3221	95	1,3	124	**1790**
22	30	900	1717	1,9	3262	25	1,3	33	**1742**
23	31	900	1726	1,9	3279	25	1,3	33	**1751**
24	32	900	1750	1,9	3325	141	1,3	183	**1891**
25	33	900	1798	1,9	3416	141	1,3	183	**1939**
26	34	900	1846	1,9	3507	141	1,3	183	**1987**
27	35	900	1864	1,9	3542	33	1,3	43	**1897**
28	36	900	1876	1,9	3564	33	1,3	43	**1909**

Also, for the same 10m-36m range (mentioned above) the evolution of the design frictional resistance on the lateral surface of the pile, the resistance on the base of the pile as well as the ultimate compressive bearing capacity are shown graphically below. This detailed analysis of the design values determined by presciptive methods is necessary to establish the actual design of the pile at the site considered.

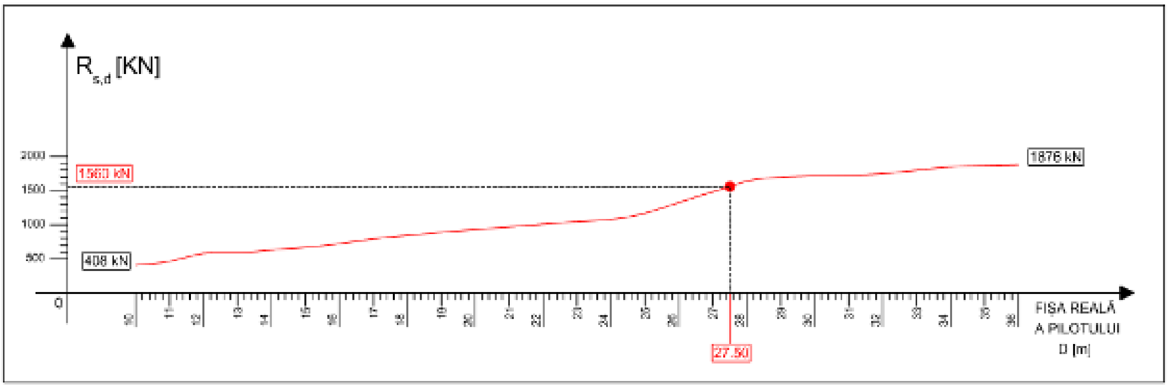


Fig.5.2_Variation of the frictional design resistance on the side surface of the pile (d=900mm) for its depth in the range of 10m-36m.

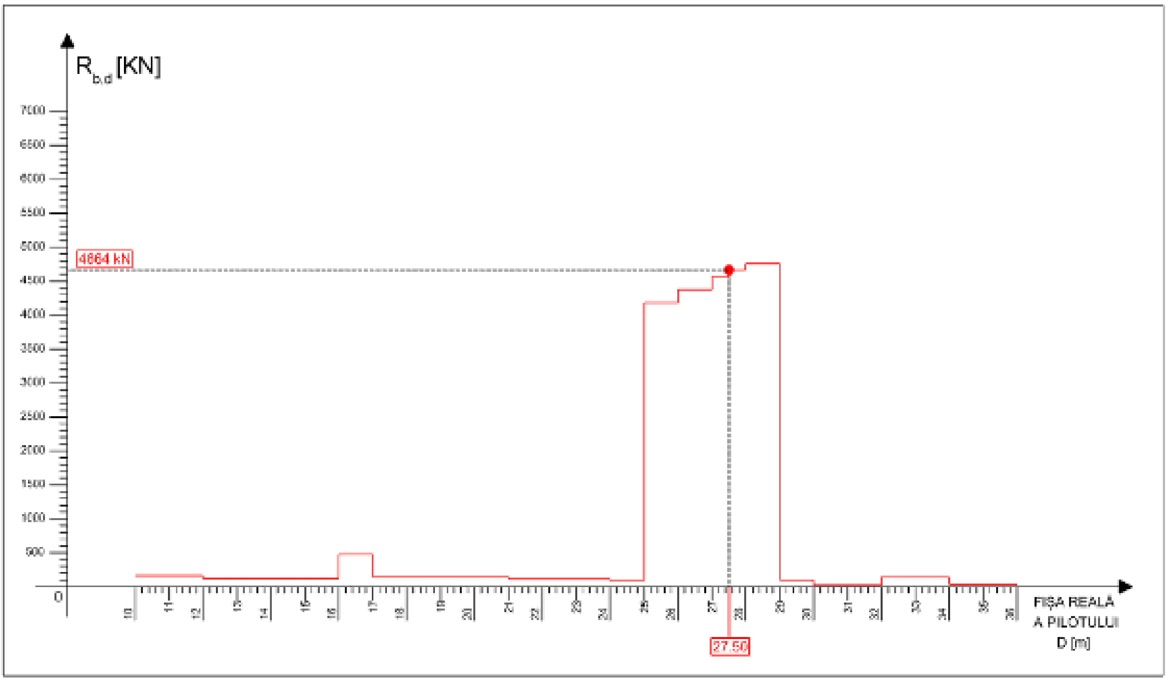


Fig.5.3_Variation of design resistance based on the pile (d=900mm), for its depth in the range of 10m-36m.

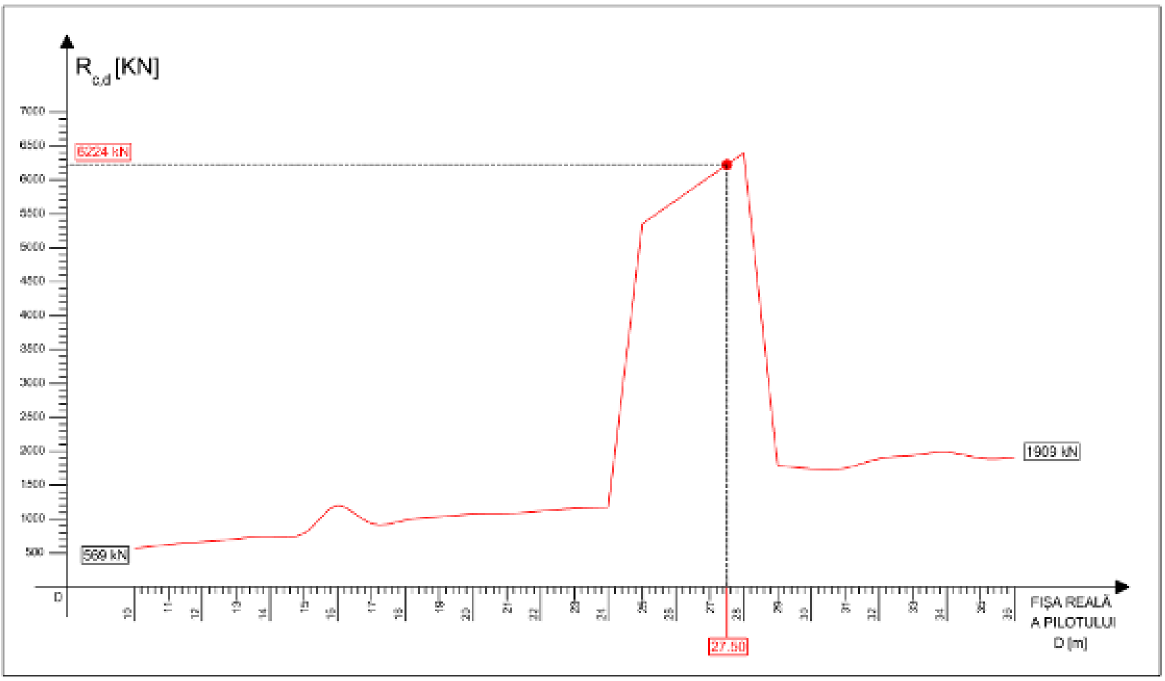


Fig.5.4_Variation of the ultimate bearing capacity of the pile (d=900mm), for its depth in range 10m-36m.

It is noted that using prescriptive methods, the results differ from one method to another but provide an optimal framework for the preliminary design of foundations in parallel with the construction and testing of test piles. Thus, for the actual determination of the design value of the ultimate compressive bearing capacity, the interpretation of the test results on the on-site test piles will be taken into account.

TEST PILES BY PERFORMING CYCLIC LOAD-UNLOAD TESTS

To determine the load-bearing capacity of reinforced concrete piles by testing, three cyclic load-unload test polygons (groups) were carried out on-site, considering a number of 8 load-unload cycles. The results of these cyclic load-unload tests (for pile Pi3, group 3) are presented in the thesis in **Annex 4_Results of** test piles.

The maximum axial force determined by analytical calculation (SLU) that can be developed in the reinforced concrete piles of the 10,900-tonne capacity cell is 6,000 KN, and the assumed value for Q_{max} = 9,000 KN. For the cyclic load-unload tests of the piles, however, a value of Q_{max} =10,000 KN was considered, while also identifying the value corresponding to the pile's ultimate load Q_r (in relation to the ground). It is taken into account that this value will be identified by reference to the resulting value of the pile settlement identified by the sum of the remaining settlements of the required cycles.

The NP 045-2000 standard addresses the issue of cyclicity only for a number of two charge-discharge cycles, of which cycle 1 limits the value of the charge to 0.5* Q_{max} , and then in cycle 2 the full value of Q_{max} materialises.

For a proper understanding of the behaviour of the grain silos studied (at the port site, which involves repeated full loading/unloading of the grain), it was necessary to take into account the full value of Q_{max} for each loading/unloading cycle.

To determine by testing the ultimate compressive load-bearing capacity of the test pile, three thresholds (values) of vertical load force, 7000 kN, 8500 kN and 10,000 kN respectively, were considered for analysis. The aim was to identify the maximum compressive bearing capacity of the pile by referring to the value of the recorded and especially to the cumulative values of the remaining loads of the loading-unloading cycles considered. The pile settlement value thus becomes the determining factor in the evaluation of the vertical bearing capacity, also taking into account that in the group behaviour of the piles, this value increases considerably and can lead to differential settlement of the grain cell roots in the studied site.

Load-unload cycle analysis for the value of the load force of 10,000 kN.

For a loading force value of 10,000 KN it can be seen that the resultant value of the remaining settlement after the 8 load-unload cycles is about 112mm. After the first four load-unload cycles, the remaining settlement had a constant character and a value of approx. 7 mm for each cycle. According to NP 045-2002, we can consider the value of 10,000 KN as the value Q_{max} corresponding to the breaking load Q_r of the pile (in relation to the ground). Particularly, the identification of the Q_{max} value was made by referring to the resulting value (by summation) of the remaining settlement, following the loading-unloading cycles considered. This could be achieved by simulating (by cycling tests on test piles) the real situation in the operation of these grain storage facilities as adequately as possible.

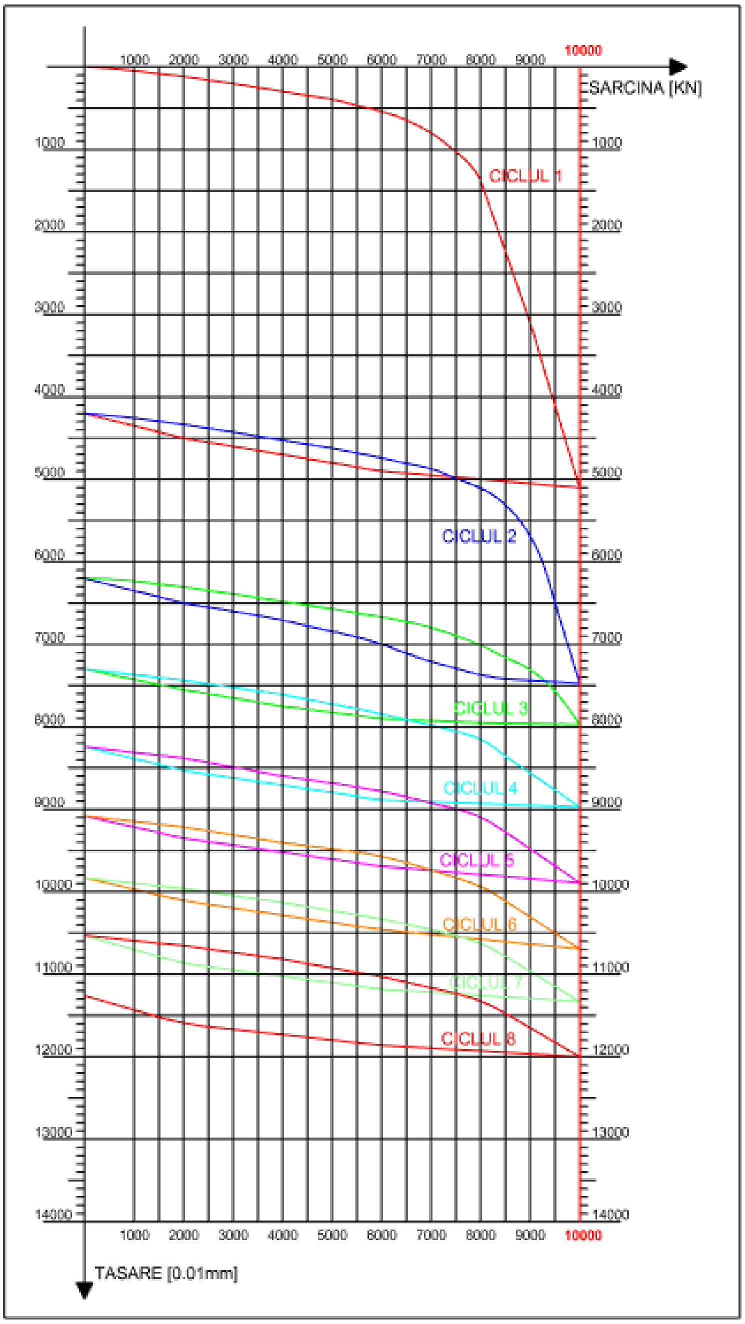


Fig 5.5_Diagram of the load-return curve (test - compression) for pile P. I3 cycle I - VIII (load Nmax = 10,000 kN).

Load-unload cycle analysis for the value of the load force of 8,500 kN.

For a loading force value of 8500 KN it is observed that the resulting value of the remaining settlement after the 8 load-unload cycles is about 32mm. After the first five load-unload cycles, the value of the remanent settlement was stabilised below 2mm for each cycle with a discharging character.

It is also found that the remaining subsidence after the completion of the first load-discharge cycle has a value of 14mm, increasing to about 32mm after the completion of cycle number 8 and there is a possibility of increase in future cycles until an "elastic" response of the foundation ground materialises.

At the same time it can be considered that the value of 8500 KN, as the characteristic value of the compressive strength ($R_{c,k}$), can be accepted considering the risk of differential settlement between the pile groups of adjacent or adjacent cell frames and the stresses/strains/deformations to be assessed when designing the infrastructure/suprastructure system (towers and steel piles).

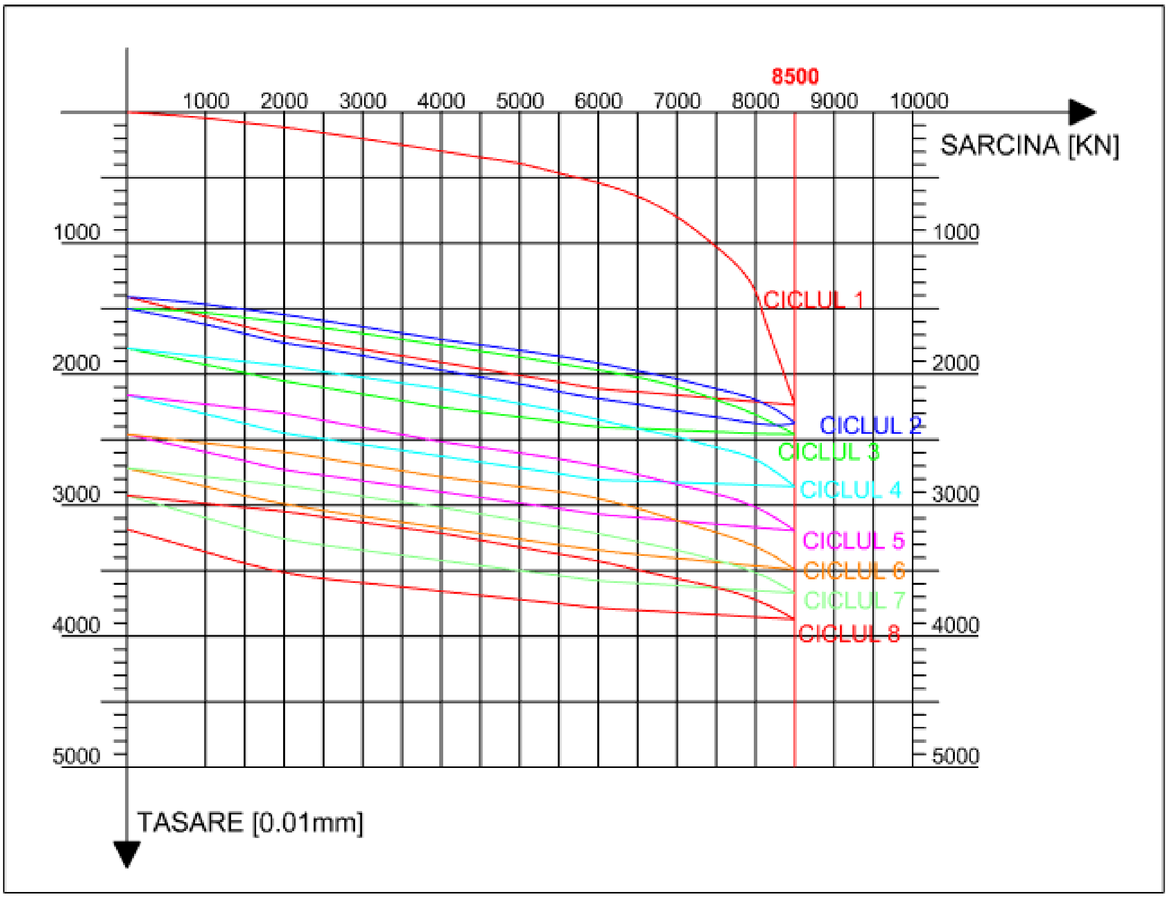


Fig 5.6_Diagram of the load-return curve (test - compression) for pile P. I3 cycle I - VIII (load Nmax = 8,500 kN).

Load-unload cycle analysis for the value of the load force of 7,000 kN.

For a loading force value of 7000 KN it is observed that the resulting value of the remaining settlement after the 8 load-unload cycles is about 2mm. After the first two load-unload cycles, the value of the remaining settlement was stabilised below 1mm for each cycle, which leads to the hypothesis of an "elastic" response of the foundation soil.

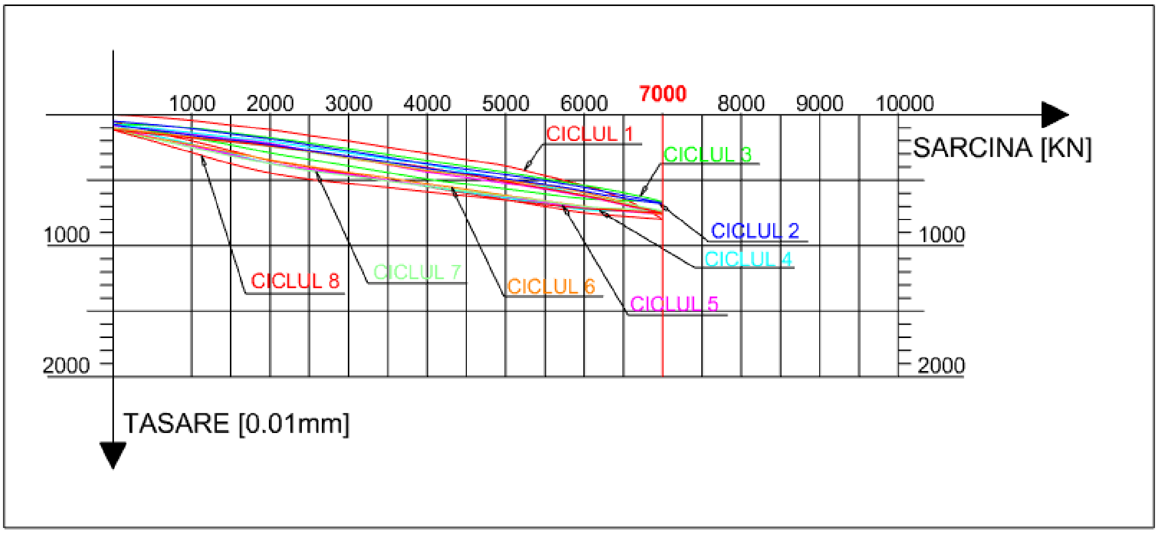


Fig 5.7_Diagram of the load-return curve (test - compression) for pile P. I3 cycle I - VIII (load Nmax = 7,000 kN).

Taking into account the results of the cyclic compression tests carried out on the test piles, the following aspects were highlighted, as follows:

- For a loading force value of 7000 KN it is observed that the resulting value of the remaining settlement after the 8 load-unload cycles is about 2mm. After the first two load-unload cycles, the value of the remaining settlement was stabilised below 1mm for each cycle, which leads to the hypothesis of an "elastic" response of the foundation soil.
- For a loading force value of 8500 KN it is observed that the resulting value of the remaining settlement after the 8 load-unload cycles is about 32mm. After the first five load-unload cycles, the value of the remanent settlement was stabilised below 2mm for each cycle with a discharging character.

It is also found that the remaining subsidence after the completion of the first load-discharge cycle has a value of 14mm, increasing to about 32mm after the completion of cycle number 8, and there is a possibility of an increase in future cycles until an "elastic" response of the foundation ground materialises.

At the same time, it can be considered that the value of 8500 KN, as the characteristic value of the compressive strength ($R_{c,k}$), can be accepted considering the risk of differential settlement and the stresses/strains/deformations to be assessed when designing the infrastructure system.

- For a loading force value of 10,000 KN it can be seen that the resultant value of the remaining settlement after the 8 load-unload cycles is about 112mm. After the first four load-unload cycles, the remaining settlement had a constant character and a value of approx. 7 mm for each cycle. According to NP 045-2002, we can consider the value of 10,000 KN as the value Q_{max} corresponding to the breaking load Q_r of the pile (in relation to the ground). Particularly, the identification of the Q_{max} value was made by referring to the resulting value (by summation) of the remaining settlement, following the loading-unloading cycles considered. This could be achieved by simulating (by cycling tests on test piles) the real situation in the operation of these grain storage facilities as adequately as possible.

From the above and taking a conservative position on the deformation of the foundation ground (also considering the size of the development of the grain terminal infrastructure as well as the particular unfavourable situations that may occur), the value of 7000 KN can be considered as the characteristic value of the compressive strength ($R_{c,k}$) of the isolated pile. The analysis of the results of the cyclic compression tests (at the action of the different thresholds of external loads, presented above) is highlighted below in **Fig. 5.8**, which highlights the limits of the pseudo-elastic behaviour of the foundation soil.

Also, the decision to avoid considering the value of 8500 KN (as a value of $R_{c,k}$) was taken taking into account the difference between its behaviour in static conditions (cycle 1 considered) and its cyclic behaviour at the end of the 8 load-unload cycles considered. In this sense, we can state that the value of $R_{c,k}$, under the effect of cyclic loads, is about 20% lower than that determined under static conditions, taking into account also the technological requirements in the operation of the grain terminal (e.g.: avoidance of differential subsidence between the groups of piles related to the adjacent cells).

Thus, taking into account the provisions of Chap. 7.4 and 7.5 of SR EN 1997-1:2004 and chapter 7 of NP 123-2010, the design bearing capacity in compression ($R_{c,d}$) will be approx. 6300 KN.

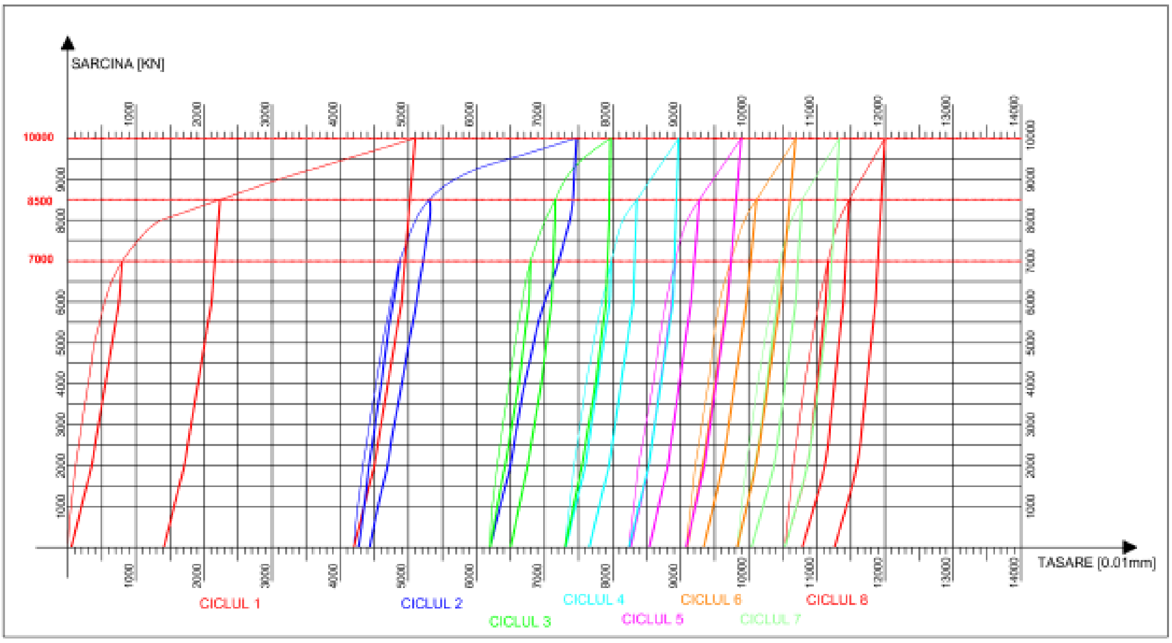
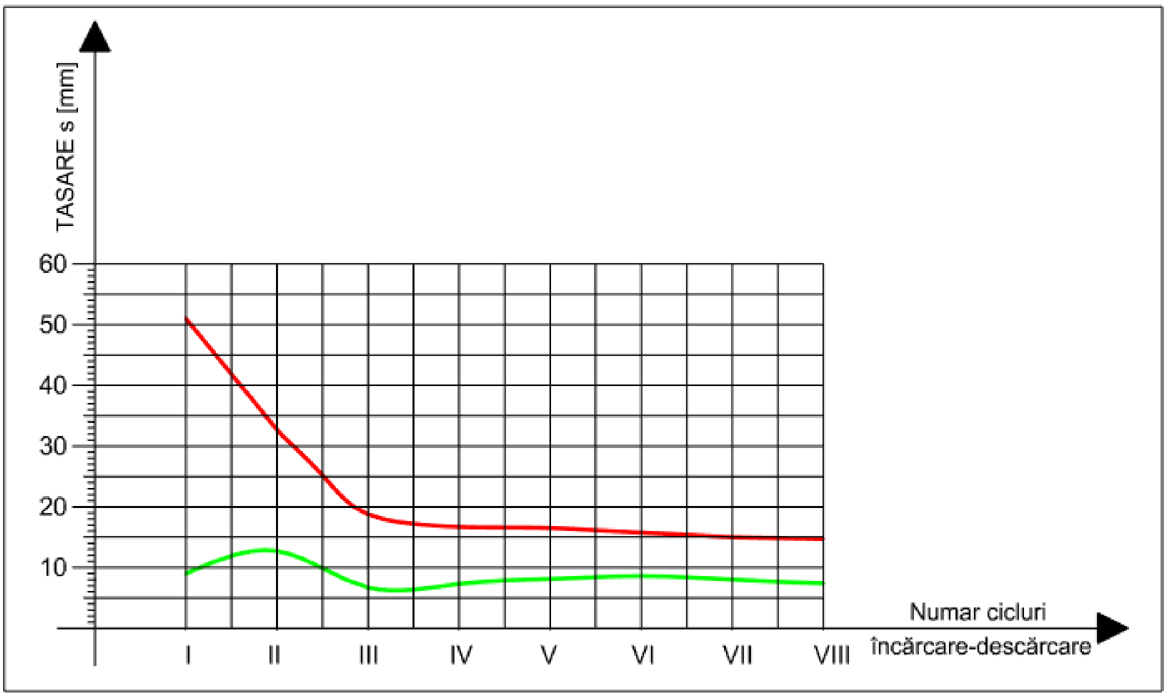
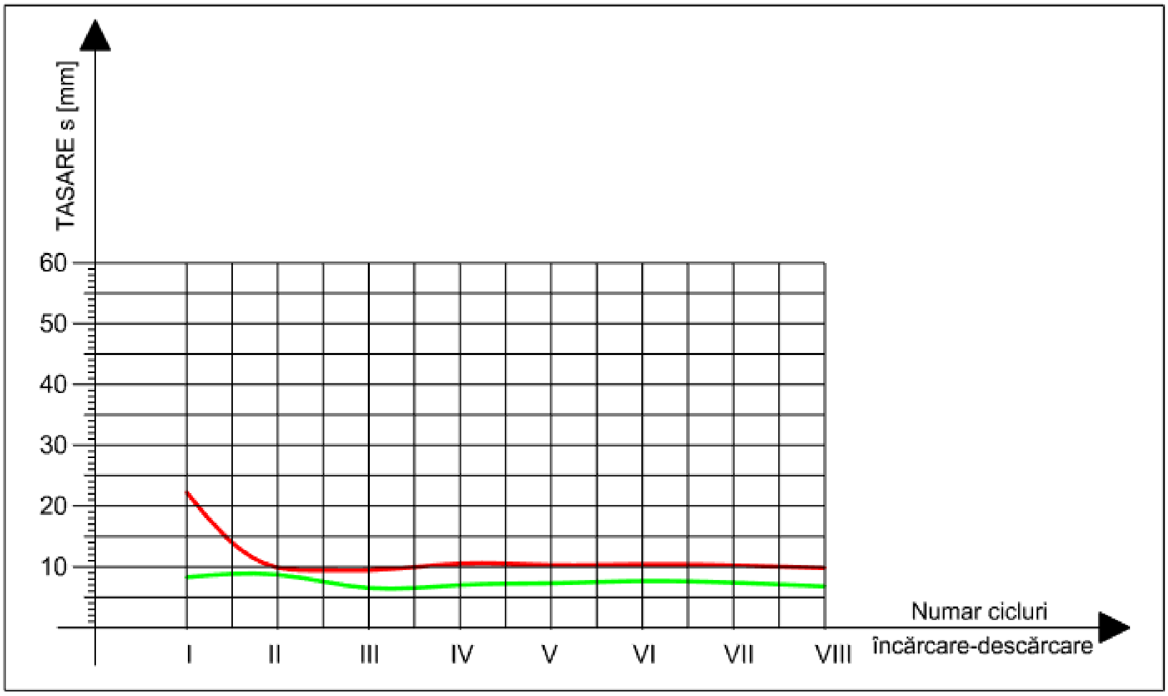


Fig 5.8_Diagram of heaps - load-return curve (compression test) Pile P. I3
Cycle I - VIII, for load Nmax = 7000 kN / 8.500 kN / 10000 kN

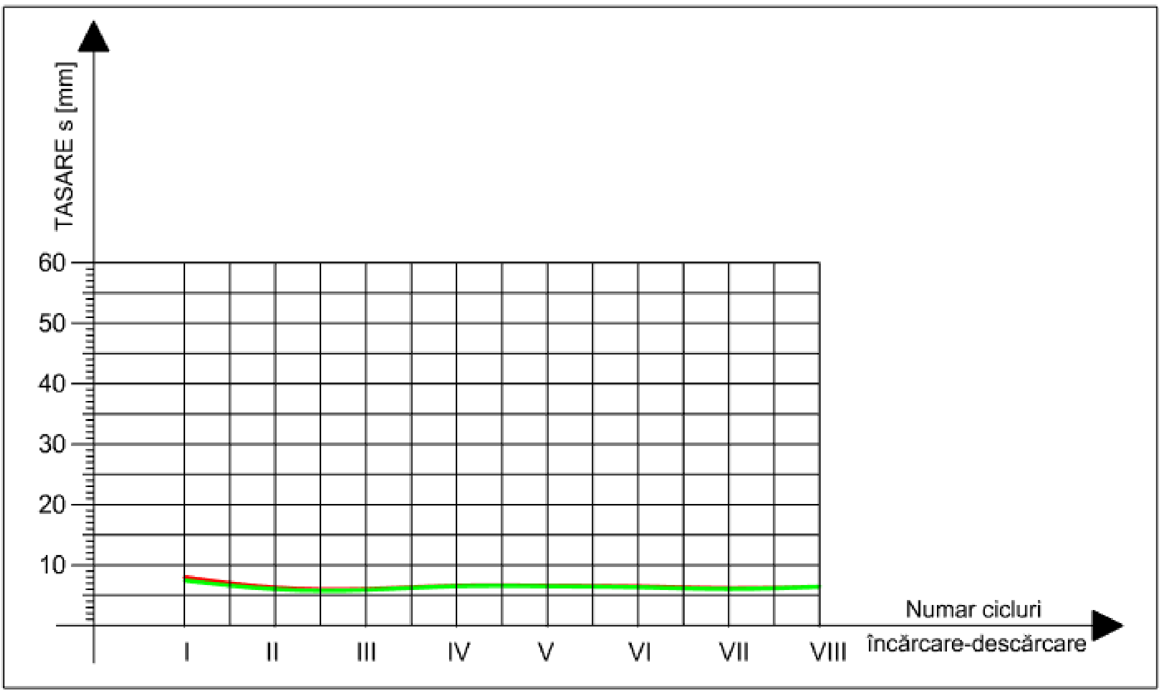
Also, from the graphical analysis shown below in **Fig. 5.9 (a,b,c)**, it can be seen that if the loading load decreases to 7000 kN the deformation is much smaller, having an almost linear character during the load-return cycles.



a) Pile diagram, load-return curve, for Pile P. I3 at load N=10,000 kN, cycle I-VIII



b) Pile diagram, load-return curve, for Pile P.I3 at load N=8,500 kN, cycle I-VIII



c) Pile diagram, load-return curve, for Pile P.I3 at load N=7.000 kN, cycle I-VIII

Fig. 5.9: Load diagram for loads of: a) 10000 kN, b) 8500 kN, c) 7000 kN

The ratio of the maximum to the return load shows that it is higher at 10000 kN load, decreases at 8500 kN load and becomes unity at 7000 kN load (**Fig. 5.10**).

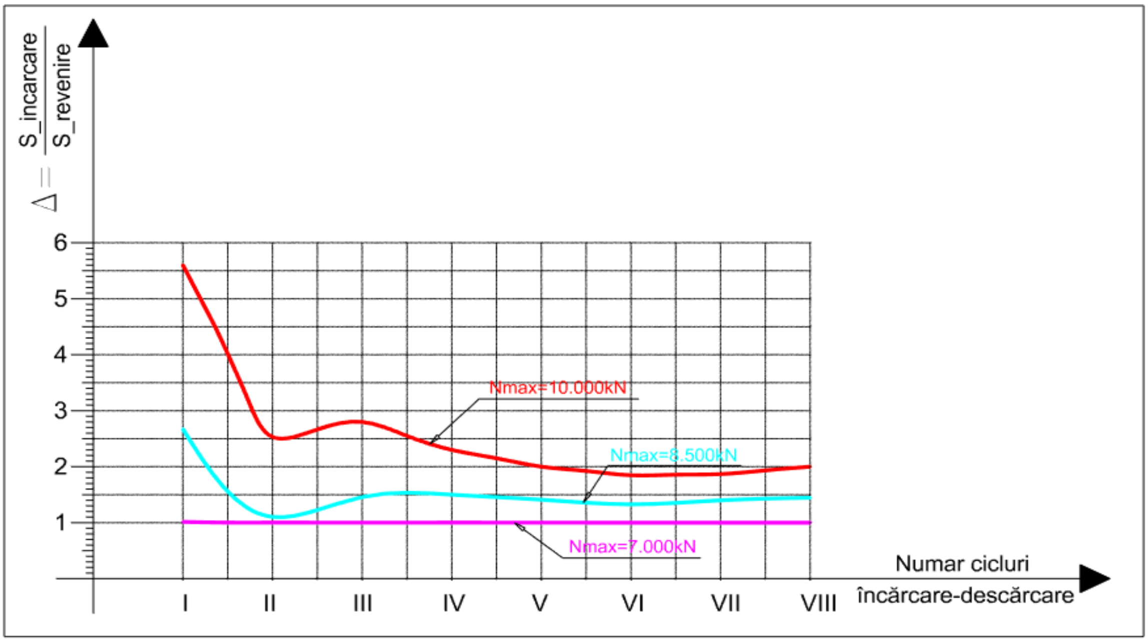


Fig. 5.10: Ratio of maximum (load) to discharge tachometer

The cumulative size of the subsidence increases with the number of loads, but it can be seen that for the 7000 kN load this evolution is zero (**Fig. 5.11**).

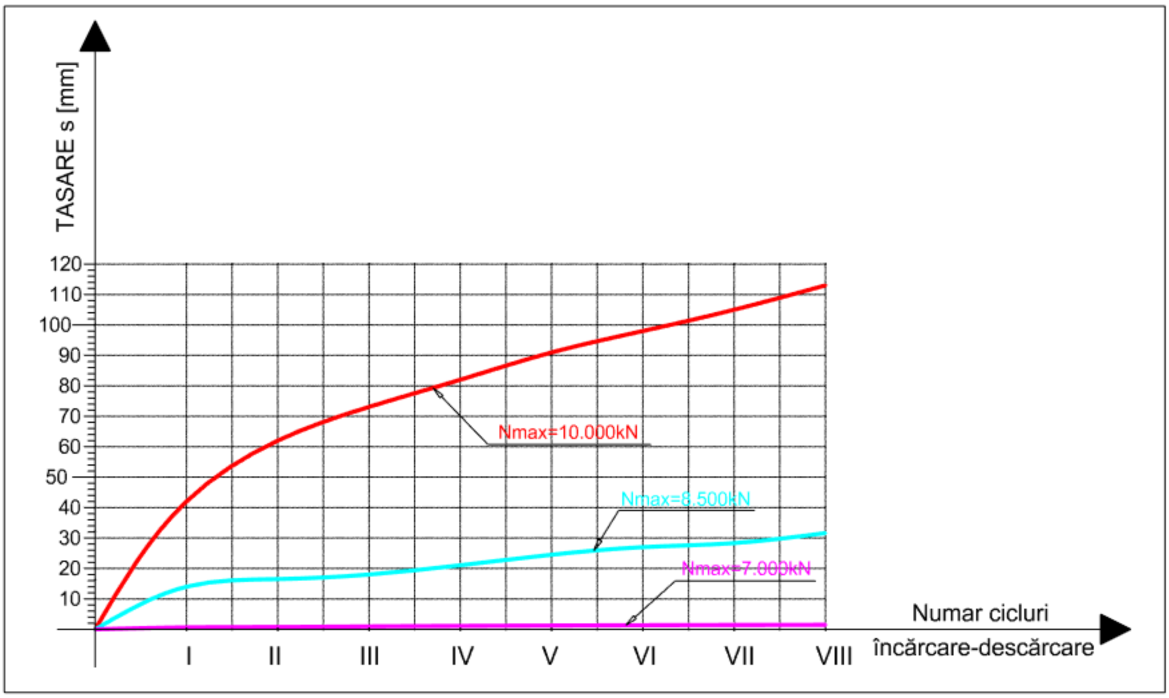


Fig. 5.11: Cumulative residual tailing diagram after 8 charge-discharge cycles

5. CONCLUSIONS AND FINAL RECOMMENDATIONS

In ports, bulk handling areas, especially grain, are set up with high silos, which offer many economic advantages (**Fig. 6.1**).

Fig.6.1 Map of grain terminals in the port of Constanta

However, there are some issues that need to be taken into account when designing such infrastructure:

- Limiting the size of the differential heights and, therefore the inclination of the structure, which is maximum 2‰ ;
- The high proportion of variable loads (grain) which is approx. 85% of the structure's own weight;
- Loading is cyclical, taking place approx. 15 loading/unloading of grain per year;
- In general the piles are floating and at high loads there is considerable settlement of the piles, corresponding to the maximum bearing capacity.

For the purposes of the above, the determination of the bearing capacity of a pile should be conditional on the lowest possible value of the resultant residual subsidence (by summation) over the loading-unloading cycles considered in the test phase of the test piles. At the limit, this value shall be lower than the threshold imposed by the limitation of the structure's deflection. This value must be considered by the designer by checking the deformation capacity (ductility) of the structural elements composing the infrastructure system in relation to the imposed stresses/deformations. Also, the determination of the value of the compressive bearing capacity of the pile shall be conditioned (in the cyclic load-unload test) also by an imposed value of the remaining settlement (usually a maximum of 2 mm) in the final cycle considered in the test.

The infrastructure supporting the structures of the metal silos is generally made of a reinforced concrete foundation founded on piles that take up and transmit the loads resulting from the grain to the ground. (**Fig.6.2**) The loading of the piles is cyclical, corresponding to the load brought by the grain loading of the silo, which is variable.

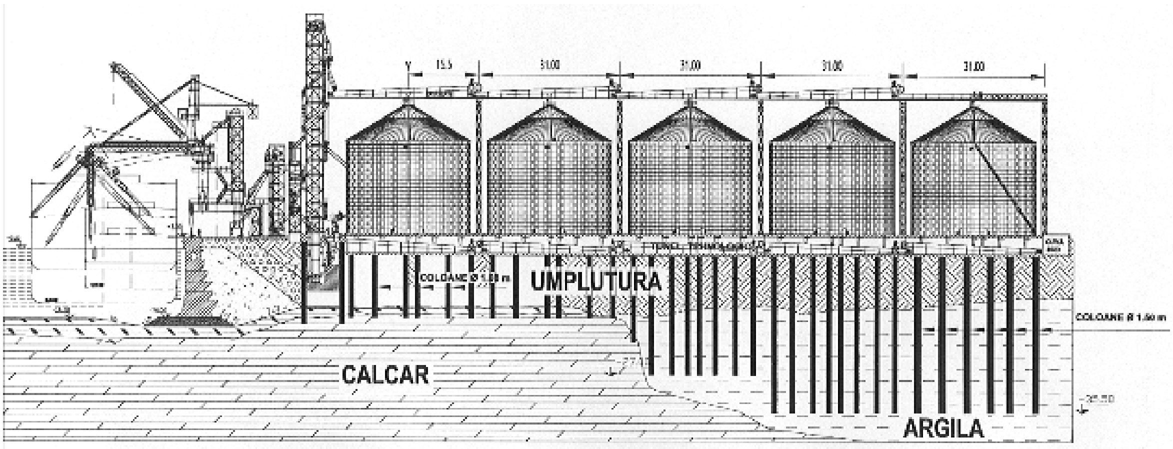


Fig. 6.2: Pile foundation of grain cells

Within the grain silo infrastructures in the port of Constanta, for the foundation of the grain cell foundations, displacement piles made of reinforced concrete were made, which in the clay soil areas are 25.0 ÷ 30.0 m long. The piles are of the floating type since the lithological sequence encountered does not include a rocky layer into which the piles can penetrate and ensure their resilience.

CONCLUSIONS

In the case of floating piles, the effect of cyclic loading must be taken into account, as the cumulative subsidence occurs, thus exceeding the permissible strength values as well as those required by the operation of technological equipment.

Thus, the cumulative residual pile settlement (after analysis of the loading-unloading cycles considered) is the determining factor that determines the bearing capacity of the pile. In this respect, the method of determining the bearing capacity (settlement) must not be limited to the provisions of the regulations in force but must be correlated with the actual behaviour of the infrastructure in the field in relation to the calculation requirements specific to each investment.

A typical example is the grain grain cells in the port of Constanta, which are loaded/unloaded over a period of 15-30 days, so the load on the piles varies cyclically.

Practical and theoretical research has shown that the effect of cyclic loads results in a permissible bearing capacity about 20% lower than under static conditions, taking into account the technological requirements for the operation of the equipment in parameters.

At the same time, in the case of the land in the port of Constanta, it is found that the deformation is considerably reduced after the first two loading-unloading cycles and then remains relatively constant, which coincides with the statistics referenced in the literature.

Since the bearing capacity of piles depends mainly on the composition of the ground, which can be very diverse, it is recommended that specific geotechnical studies and tests, including cyclic tests, be carried out for each site, at least for objectives with a high variable load and a relatively small percentage of the structure's dead weight (permanent loads) in relation to the live loads considered.

The specific nature of the foundation soil in port areas, which consists of fills, then sands and clay formations, and at depth limestone in various states of degradation, requires that in the case of grain terminals, the foundation piles should be checked for cyclic actions in view of the variability of live load. The studies and calculations carried out have shown that in the field conditions in the port of Constanta, taking into account some technological constraints, the design bearing capacity of piles at vertical loads is approx. 20% lower for cyclic loads than for static loads, which is important to take into account when designing infrastructure solutions.

PERSONAL CONTRIBUTIONS

The subject of the PhD thesis, concerning the behaviour of foundations on piles, axially stressed cyclically with low frequency in the port infrastructure of Constanta (within the infrastructures of grain warehouses and not only), is complex because of its unique and specific character. The restricted and limited framework of regulatory provisions in this regard creates difficulty in determining the ultimate bearing capacity of this type of pile, and the main personal contributions made in this direction include:

1. Materialisation of a current bibliographical synthesis, as a result of the browsing of scientific documentation in the field of reference, most of which is only available in the international literature. These materials include experimental research on specific investments, design standards and regulations, analytical calculation methods, prescriptive methods, numerical and empirical or analytical calculation models, and user manuals.
2. Issuing practical recommendations for the design of pile foundations under low-frequency vertical cyclic loading, based on the experience gained during the design of the Port of Constanta, in comparison with the current normative provisions that do not cover this particular type of construction behaviour. These recommendations can be

extremely useful for designers and can form the basis for completing the provisions of the current regulations. It is recalled that in SR EN 1997-1 there is no reference to the use of prescriptive methods in the case of axially loaded piles (see also subchapter. 7.5.1.Piles exposed to axial loads - Compressive bearing capacity in GP 129/2014 - Geotechnical design guide). In this sense, the use of these methods should only be of an indicative and pre-dimensioning nature in the design of pile foundations.

3. Proposals to supplement the provisions of the current regulations on the load-bearing capacity of piles subjected to low-frequency cyclic loads.

4. Application of the results of the studies and research to some major objectives in the port of Constanta (large grain warehouses) where the loading on the foundations has a cyclic character during operation/operation.

5. Determination of the effect of cyclic loads on piles, assessing the limit below which their effect is low and can be neglected.

During the whole research activity, as author and co-author, a number of scientific articles have been elaborated and published, as follows:

1. Tsitsas, G., Ciortan, R., **Dogaru**, P., Miritoiu, P.: "On the waterfront: case histories from soil improvement works." Geo-Environmental & Construction European Conference, Tirana, Albania 2015.
2. **Dogaru**, P., Ciortan,R.,Tsitas, G.: "Foundation solutions for grain warehouses in the Port of Constanta" at the 13th National Conference on Geotechnics and Foundations - Cluj - Napoca - 7-10 Sept. 2016 - pg 349-356.
3. **Dogaru**, P., Krkljus, D., Vlăescu, D.: "Proper infrastructure design at high vertical loads, near waterfront structures" published in Ovidius University Annals Constantza (OUAC) - Series Civil Engineering, Year XXI (2019), Issue XXI (http://revista-constructii.univ-ovidius.ro/doc/editii/2019.pdf).
4. Vlăescu, D., **Dogaru**, P., Krkljus, D.: "The specific use of composite materials in the consolidation of historical monuments", published in Ovidius University Annals Constantza (OUAC) - Series Civil Engineering, Year XXI (2019), Issue XXI (http://revista-constructii.univ-ovidius.ro/doc/editii/2019.pdf).
5. **Dogaru**, P., Krkljus D., Ghencea, A., Tudose, C., Didă, L., "The new grain terminal at berth 80, Constanta South Port - the challenges and outlook of a great project". Romanian AICPS journal Review 3-4/2019. (http://www.aicps.ro/revista/aicps-review-3-4-2019/content#page/n59/mode/2up)
6. Ciortan, R., **Dogaru**, P., Tsitas, G., "Increasing Grain Traffic through the Port of Constanta" - at the ASTR Days Conference - 14th Edition 17-18 Oct 2019 -Chisinau, Republic of Moldova.

7. **Dogaru**, P., Ciortan, R., "Behaviour of piles under the effect of vertical cyclic actions" in the 14th National Conference on Geotechnics and Foundations - Bucharest, 2-3 June. 2021.

FUTURE RESEARCH DIRECTIONS

Taking into account the complexity and difficulty of the research topic (pile foundations required vertically cyclically at low frequencies), the following research directions can be considered, as follows:

- Analysis of the cyclic stress behaviour of a foundation in different types of soils knowing that a clay massif behaves completely differently from a sandy one.
- Identify, through geotechnical laboratory studies, the response of the ground to cyclic behaviour of the actions, given the much lower costs than real tests on test piles. In this sense, by exposing different soil samples (in the laboratory) to cyclic stresses of different intensities, a picture of ultimate bearing capacities can be created to provide expert designers with the first intention of pre-sizing infrastructures on piles vertically stressed cyclically with low frequency.
- To create research directions in an important area of pile foundations, which can be the subject of other PhD theses.

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