sciendo

Studia Geotechnica et Mechanica, 2021; 1–13

Original Study

Open Access

Paweł Więcławski*

Vibro piles performance prediction using result of CPT

https://doi.org/10.2478/sgem-2021-0024
received October 4, 2019; accepted September 11, 2021.

Abstract: Vibro piles belong to the group of full displacement piles with an expanded base, characterised by a very high load capacity, especially in non-cohesive soils. The problem is to adopt a reliable method for the determination of full load–settlement (Q–s) curve. A frequent difficulty is the determination of the load capacity limit based on the static load test because the course of the load–settlement curve is of a linear nature. This publication presents the empirical method. It allows direct prediction of a full axially loaded pile settlement curve based on the values of q_c cone resistance obtained in cone penetration test (CPT). The advantage offered by this procedure is the accuracy of the obtained limit values in relation to the actual load-bearing capacity as compared to other methods based on soil parameters obtained in in situ testing. An additional advantage is the Q–s characteristics, which enable designing for intermediate values, allowing for the criterion of minimal or equal settlements. The shape of analytical curves was compared with static pile load test (SPLT) curves. This comparison showed large convergences between the analytical and measured curves.

Keywords: Vibro Piles; Ultimate Capacity; Settlement Curve; CPT Probe.

1 Introduction

In their present form, Vibro piles were implemented as early as in the 1960s. They are installed by driving in a steel pipe with a closed end using a diesel or hydraulic hammer. Then, reinforcing is inserted into the pipe inside and the pipe is filled with concrete. Extraction of the pipe using a vibrator causes compaction and good integration of the pile shaft with the ground. An additional advantage of the technology is the short time necessary for installation in diverse soil conditions, and concreting in a casing pipe ensures pile continuity [1-2].

Pile designing frequently makes use of the results of in situ testing. These are primarily the parameters determined based on a cone penetration test (CPT), dynamic probe testing and testing using dilatometer and pressuremeter [3-5]. These methods are among the group of direct methods and have been based on empirical dependencies. The CPT is particularly useful for designing pile foundations [1, 6]. Probing parameters q_c and f_s obtained from the CPT are directly used to determine unit resistance under the base and on the pile side surface. In most cases, empirical correlations are adopted, confirmed by the results of static pile load tests on a natural scale. Calculation methods of pile load bearing differ in the ways by which q_c values are averaged and by correlation coefficients that depend on the type and state of soil and the method of pile installation [7-11]. A special place belongs to methods based on transfer functions. The calculations are based on non-linear load transfer functions of $t - z$ type for shaft and $q - z$ type for base resistance of pile. Pile deformation is described in terms of elastic solutions, whereas load–settlement relationship is determined by iterative procedure. Load-transfer functions method enables the evaluation of entire range of settlement curves using the results of CPT [9, 12-13].

To sum up, the idea of transformation functions is, therefore, to determine the course of the pile settlement curve to the previously calculated bearing capacity. This curve is usually a hyperbola from the starting point up to a conventional limit value. The new curve prediction method for Vibro piles, presented in this work, is of a completely different nature. Based on the developed correlations, the full load–settlement characteristics for the analysed pile are determined first. The next step is to read off from this curve the appropriate value of bearing capacity which corresponds to the assumed limit settlements. It is also possible to determine the settlement that corresponds to the design loads. The shape of the settlement curve is an important element. For the procedure presented in this article, the curve is described by two functions: linear and exponential. The linear part, which is characteristic

***Corresponding author: Paweł Więcławski,** Gdańsk University of Technology, 11/12 Gabriela Narutowicza Street, 80-233 Gdańsk, Poland, E-mail: pawwiecl@pg.edu.pl

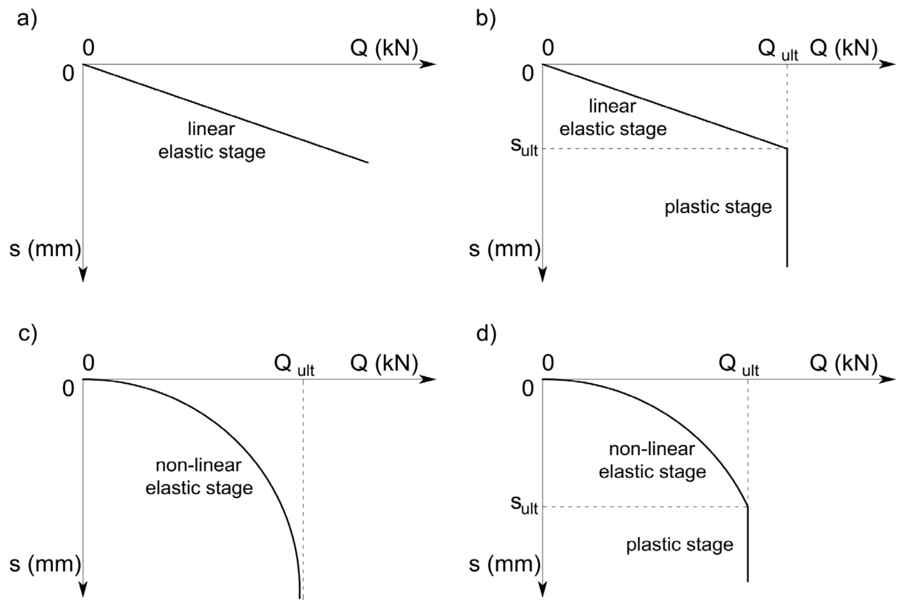


Figure 1: Theoretical model of pile and soil interaction: a) perfectly elastic; b) linear elastic and plastic; c) non-linear elastic; d) non-linear elastic with plastic yielding.

of Vibro piles due to the technology and the excellent interaction of the pile with the soil, describes the elastic deformations. This part of the curve is generated when the resistances on the lateral are mobilised. The non-linear part is described by an exponential function. It describes the elastic–plastic phase in which there is a mobilisation of resistance under the pile base up to a boundary value. The combination of these sections, linear and non-linear functions, gives us a true picture of the pile–soil interaction as the axial vertical load increases.

In many CPT-based methods, use is made of high global safety factors (F_S). For the methods applied in Europe, F_S ranges from 1.5 to 3.0 (e.g. methods of Bustamante - Gianeselli or Gwizdała -Stęczniewski) [14-15], depending on the soil conditions, technology of piles, type of structure, etc. Another approach to methods based on the soil test results has been included in the standard PN-EN 1997-1:2008. When calculating the characteristic value of load-bearing capacity, use is made of correlation coefficients ξ_3 and ξ_4. Passing on to design values, the characteristic value is divided by the relevant resistant coefficient, which depends on the design approach [16]. In order to determine the computational curves of Vibro pile settlement, which were determined using the presented procedure, an approach according to Eurocode 7 was proposed. The use of a set of correlation coefficients and partial safety factors guarantees safe values of the pile-bearing capacity. The global safety factor Fs = 1.3 was also used for comparison. This is the minimum value of the coefficient, which also provides safe estimate values of the design vertical bearing capacity of Vibro piles.

2 Concept of theoretical curve Q–s for Vibro piles

Based on the existing theoretical models for pile interaction with the soil (Fig. 1) [17-19] and 137 settlement curves obtained from static load tests, from various locations in Poland (Table 1), the Q–s characteristics dedicated to the Vibro pile technology have been defined (Fig. 2).

The data comes from geotechnical and geological-engineering documentation created for real engineering structures such as road viaducts, bridges, tanks, harbour quays and industrial halls. The analysis concerns pile diameters 408, 457, 508, 560 and 610 mm with lengths from 6 to 22 m. The relationships presented in the next section are based on over 100 pile test results and nearly 90 CPT static soundings. The correlations shown are concerned with piles whose base is in the layer of fine and medium sands. Piles terminated in coarse sands were omitted due to the small set of cases. However, they provide an excellent reference for the relationships obtained and confirm the correctness of the developed method. This is because the curves from static test loads for these piles are better than the curves for piles in medium and fine sands.

The developed procedure and the adopted theoretical assumptions enable determination of the full Q–s curve

Table 1: Compilation of data used for correlation analysis.

Location	Number of piles	Geometry of piles			Number of CPTs	Distance between pile and CPT	Soil under pile base
		D	D_b*	L			
		(mm)	(mm)	(m)		(m)	
Gdańsk	5	406	460	9.0 to 13.5	6	2.0 to 10.0	FSa
	9	508	560	16.0 to 22.0	7	2.0 to 10.0	FSa
Gdynia	14	560	610	14.0 to 18.0	10	2.0 to 6.0	FSa
Grudziądz	16	508	560	10.6 to 20.0	12	1.0 to 8.0	FSa, MSa
Hajnówek	4	508	560	7,5	4	4.0 to 6.0	CSa, FSa
Olsztynek	44	610	660	6.5 to 12.0	30	1.0 to 6.0	MSa
Szczecin	13	408	460	12.0 to 14.0	6	2.0 to 10.0	MSa, FSa
	6	457	510	17.6 to 18.4	6	1.0 to 8.0	FSa
	10	457	520	17.5 to 21.5	9	1.0 to 6.0	FSa
Wrocław	4	610	660	8.5 to 11.0	4	2.0 to 6.0	CSa, MSa
	12	508	560	6.0 to 18.0	8	2.0 to 6.0	CSa, MSa
S	137				102		

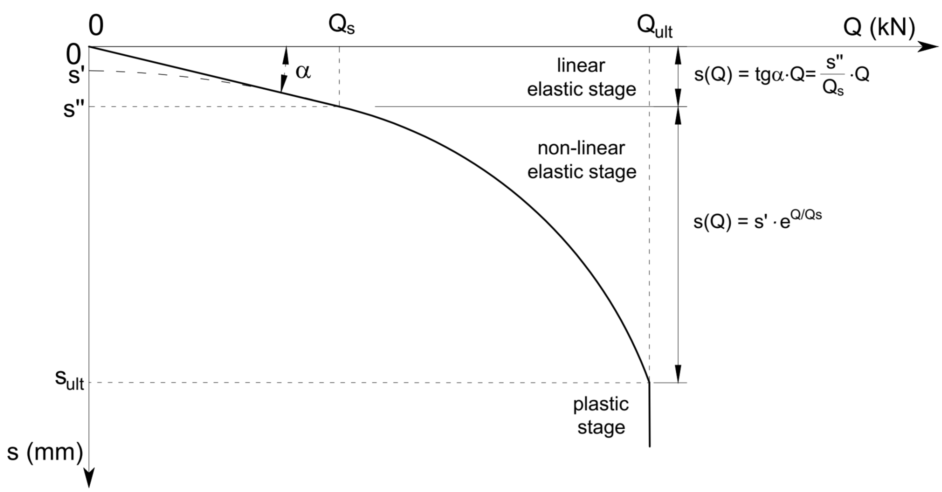


Figure 2: Theoretical model of load–settlement curve.

to identify the value of load-bearing capacity and indirect (Q_i, s_i) or ultimate (Q_{ult}, s_{ult}) settlements. According to the guidelines of the currently valid standard PN-EN:1997-2008 [16, 20], the load-bearing capacity threshold value is adopted for settlements of 0.1D (10% of the pile diameter).

Pursuant to the adopted assumptions, the elastic stage of pile interaction with the soil has been defined by the following dependence:

$$s(Q)=tg\alpha\cdot Q=(s''/Q_s)\cdot Q \quad (1)$$

where: tgα = directional coefficient of linear function, describing the linear elastic phase of pile–soil interaction; Q = vertical load with axial impact on pile (kN); s” = value of pile settlements in the transition point of curve Q–s from the linear elastic stage into non-linear elastic–plastic stage of pile–soil interaction (mm) and Q_s = value of applied load causing settlement s”, load initiating the elastic–plastic stage of pile–soil interaction (kN).

The non-linear elastic phase is determined with the following dependence:

$$s(Q)=s'\cdot e^{Q/Qs} \quad (2)$$

where: s’ = initial point of the exponential function for Q = 0 kN (mm) and e = 2.718282, Euler’s number.

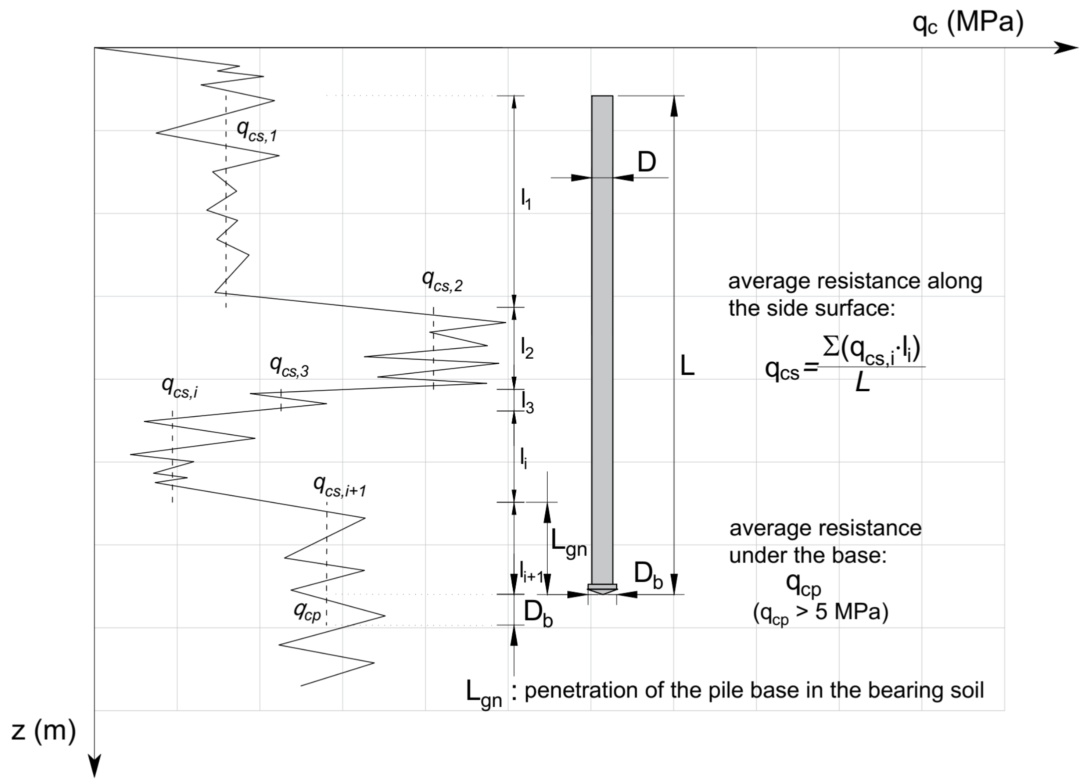


Figure 3: Template of averaging resistance from CPT probing.

3 Procedure for determination of theoretical Q–s curve based on CPT results

Key elements that affect the form of the Q–s curve and the form of a function that describes this curve are the parameters of the subsoil. As the adopted method is a proposal of the direct method based on CPT probing, it requires appropriate averaging of resistance q_c measured on the penetrometer cone. Methods of averaging the resistance q_c are presented in Fig. 3. The following were considered as being of importance:

– averaged resistance q_c along the pile side surface – q_{cs}, within particular soil layers;
– averaged resistance q_c under the pile base – q_{cp} to the depth of $1D_b$, where D_b is the pile base diameter, for Vibro piles: diameter of steel shoe (the base should be in the soil with qc > 5 MPa) and
– determination of pile base penetration in bearing soils – L_{gn}, measured from the pile base to the lower surface of weak soil.

For the proposed method, of particular importance is the strict empirical correlation between ground susceptibility s“/Q_s, which determines the form of the Q–s curve, and soil parameters around the pile, that is, the mean value of cone resistance of the CPT along the side surface – q_{cs} and under the pile base – q_{cp}. The dependence has been defined on the basis of an analysis of no-linear regression between soil parameters and those of Q–s characteristics. A graphic representation of the dependencies is presented in Fig. 4.

Further parameters necessary to define the Q–s curve, which fully depend on ground susceptibility, are as follows:

– initial point of an exponential function – s’ and
– transition point of the linear phase into a non-linear one, that is, the initial point of curvature – s”.

Based on the actual curves obtained from 50 static load tests of the piles, it has been found that the settlement value s’ should be determined on the basis of ground susceptibility s“/Q_s, allowing for the type of soil under the pile base (Fig. 5).

There is a specific constant linear relation between the settlement values s’ and s” [2] (Fig. 6) and this relationship results from the continuity of the linear and non-linear functions s(Q).

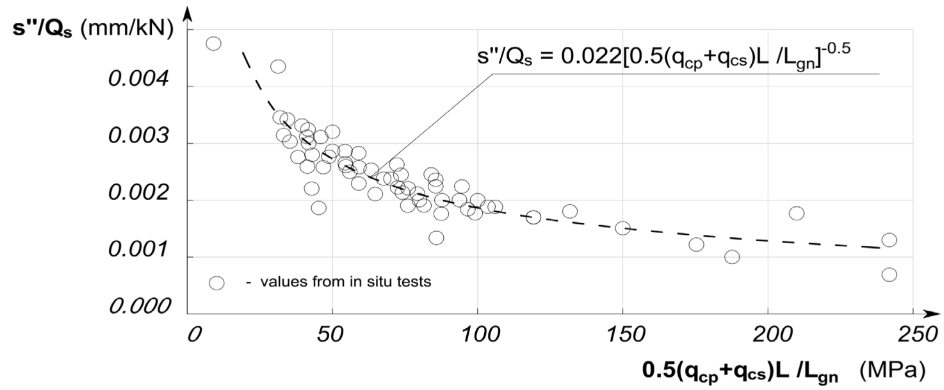


Figure 4: Dependence of ground susceptibility on parameters of the soil and pile geometry.

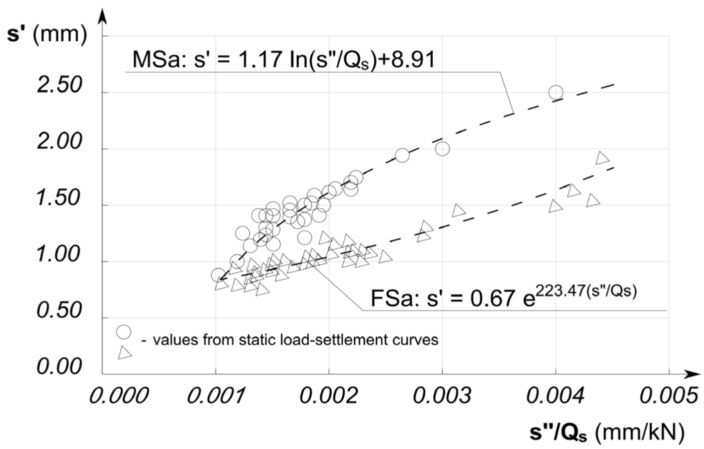


Figure 5: Dependence of the initial point of the exponential function s' on ground susceptibility.

If the values s' and s" are available, it is possible to set out the value of Q_s, load causing settlement s", which initiates a non-linear elastic phase of the pile–soil interaction.

4 Application of procedure for design needs

The advantage of using EC7 is the possibility of applying, for design and verification needs, any method applied in national practice. The consistency of calculations depends to a great extent on the reliable determination of unit resistance under the pile base and on its side surface. Many calculation methods exist that may be adopted to perform practical calculations. The adoption of a reliable method should be based on comparative analyses and the adoption of statistical methods that utilise results obtained from static load tests.

Below is an outline of the determination method of Q–s characteristics in accordance with the developed procedure for a Vibro pile having a diameter of 457/520 mm and length of L = 18.5 m, installed in fine sand.

On the basis of a standard set of data necessary to design a pile foundation (Fig. 7), such as soil parameters (in this case, the cone resistance from CPT) and the geometric parameters of pile, the full settlement curve has been determined. The presented empirical correlations (Figs 4–6) enable the definition of values that characterise particular stages of pile interaction with the soil, and specifically the linear elastic stage (Fig. 8a) and the non-linear one (Fig. 8b), determined by dependencies (1) and (2) and the theoretical point of soil yield under the pile base (Table 2). A synthesis of these relations is a prediction of the pile–soil interaction depending on the applied axial load (Fig. 8c). The best method for verifying the accuracy and reliability of analytical methods includes tests performed on a natural scale in real soil conditions. Fig. 8d presents a

Table 2: Output parameters according to correlations from Fig. 4 to Fig. 6.

s''/Q_s mm/kN	s' mm	s'' mm	Q_s kN
0.0042	1.73	4.7	1107

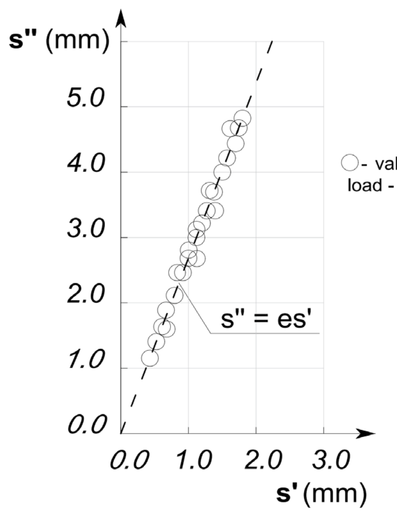


Figure 6: Relationship between the directional coefficients of the linear function s'' and the non-linear function s'.

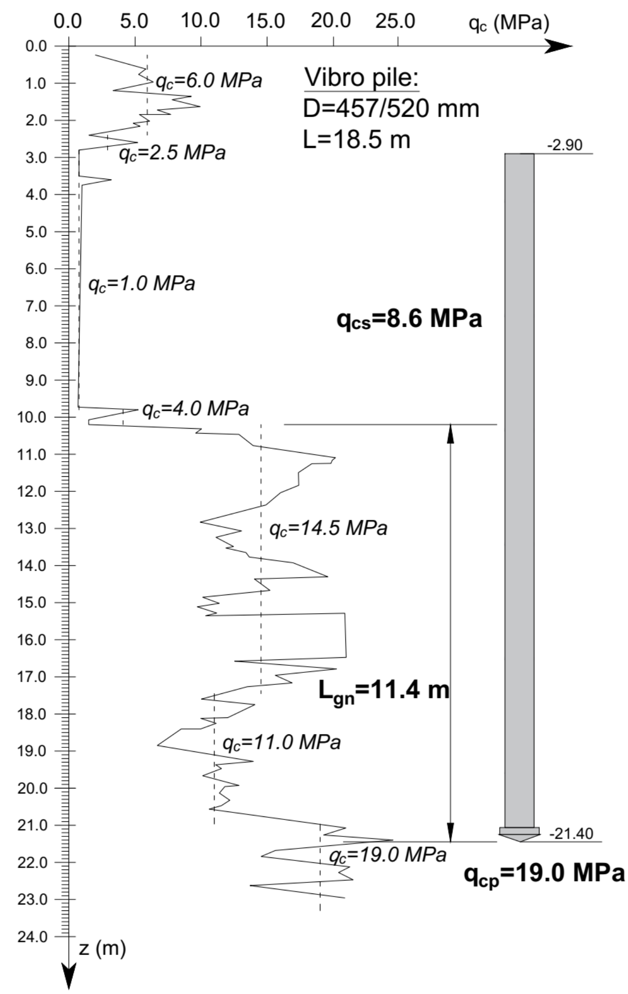


Figure 7: Results of probing and averaged parameters from CPT testing.

comparison of the theoretical settlement curve with the curve from static load test.

Below are given compilations of resistance q_c (Fig. 9) and curves (Fig. 10) from different projects from Gdansk (Pile No. 1–3), Szczecin (Pile No. 4–6) and Grudziadz (Pile No. 7–9). The piles vary in diameter and length and their bases are in fine or medium sand.

Full agreement of the analysis results with the actual bearing capacity is not obtained in every case. Sometimes, we get slightly higher values, and sometimes slightly lower values.

In order to use the theoretical curve Q–s for design purposes, it is suggested to use the safety factor of the minimum value of Fs = 1.3, taking into account the following:

- errors resulting from averages of individual parameters in the data generation phase;
- variability of soil conditions in relation to the sounding results;
- distance of piles from CPT sounding, which is sometimes equal to approx. 10 m;
- technological factors of pile construction;
- influence of time on soil condition and bearing capacity increment and
- statistical analysis errors.

The factor Fs = 1.3 provides a safe range of values of theoretical bearing capacity for all analysed piles. If the calculation method with too high uncertainty has been adopted, it is allowed to introduce a design model factor into the analyses to correct for systematic errors and to ensure that the estimated pile-bearing capacity is more safety.

The recommended computational path for the proposed method assuring better security is the methodology proposed by EC7 [16]. For piles and pile foundation, the adopted computational approach is DA2. A schematic diagram of the system of calculation curves after application of particular safety factors is shown in Fig. 11.

Values of computational load-bearing capacity $Q_{ult,d}$ ($R_{c,d}$), according to EC7, are determined by adopting the following:

- correlation coefficients ξ_3 and/or ξ_4 for determination of load-bearing characteristic values based on the results of soil testing ($R_{c,k}$) and

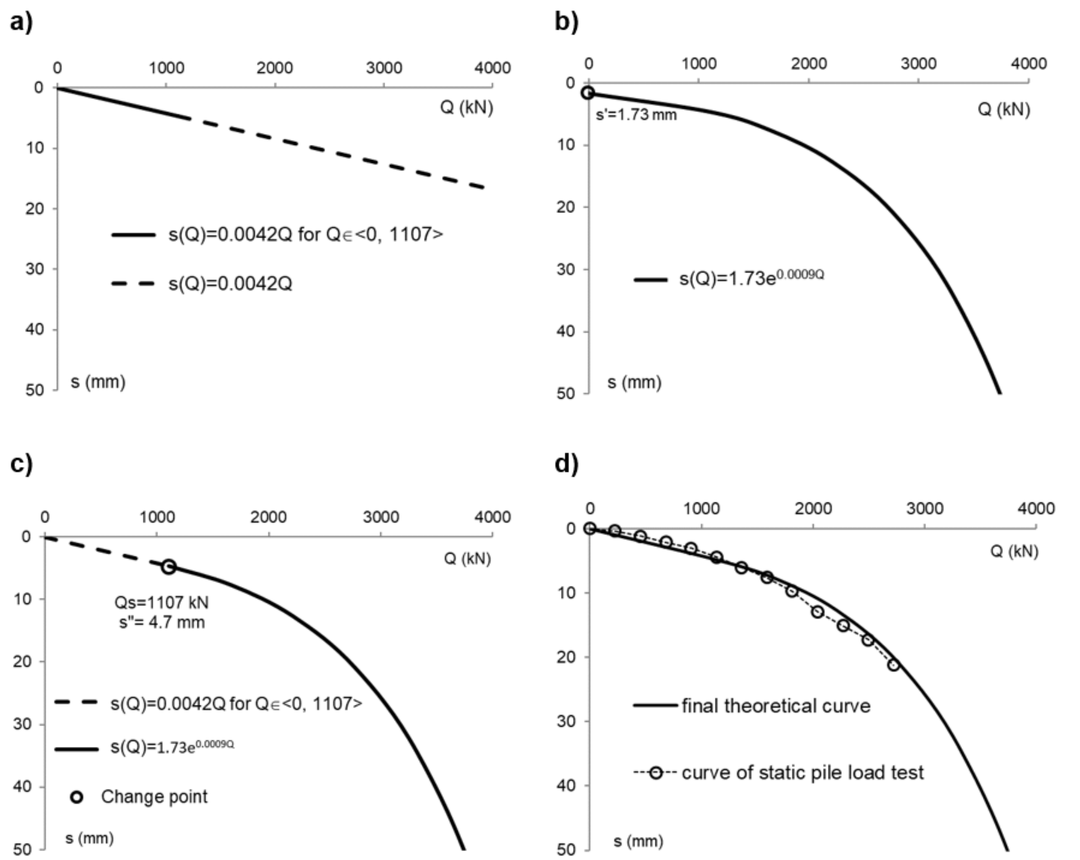


Figure 8: Stages of Q–s characteristics prediction based on the proposed procedure: (a) linear elastic stage; (b) non-linear elastic stage; (c) combination of the linear elastic stage for Q□<0, Q_s> and non-linear elastic stage; (d) comparison of the theoretical curve with the measured curve from static load test.

– partial coefficients for load-bearing capacity γ_R ($R_{c,d}$) (Fig. 11).

In Fig. 12 are presented computed curves with the use of coefficients Fs = 1.3 and according to EC7 and the curves from SPLT tests for the most unfavourable cases from Fig. 10. Theoretical curves run for pile No. 4 and pile No. 7 above the actual curves. In the other cases presented, the predicted curves overlap or are under the curves from the test loads. In both cases (pile No. 4 and pile No. 7), dividing the load value by the factor Fs = 1.3 corrects the overestimated values. The use of the DA2 approach according to EC7 makes the calculation curves determined from the procedure very safe. They will definitely ensure safe cooperation of the pile with soil and fulfilment of the ultimate limit state condition.

Design values $Q_{ult,d}$ are a safe estimation of load bearing, given diverse methods of interpreting static load test. As has been mentioned already, Vibro piles are characterised by very good load-bearing capacities in non-cohesive soils, hence the nature of Q–s characteristics is quite similar to the linear one. Determination of the ultimate value requires a curve extrapolation. Fig. 13 contains a reference to analytical values of $Q_{ult,cal}$ to limit the load-bearing capacity determined on the basis of curve interpretation obtained from SPLT by the Chin's method [21]. The Chin's method is based on the assumption that the Q–s curve for a pile is approximately hyperbolic.

From the above graph, it can be seen that the characteristic values of the limit load based on the proposed method are close to the results obtained by Chin's method. The difference between these values is much smaller than the relationship between the values from Chin's method and those calculated using the direct method according to EC7. It should be emphasised that the method included in EC7 gives higher values and closer to real pile-bearing capacities than many other methods used in Europe and in the rest of the world, for example, LCPC, Gwizdała–Stęczniewski's method, Schmertmann's method, De Ruiter and Beringen's method, Aoki and

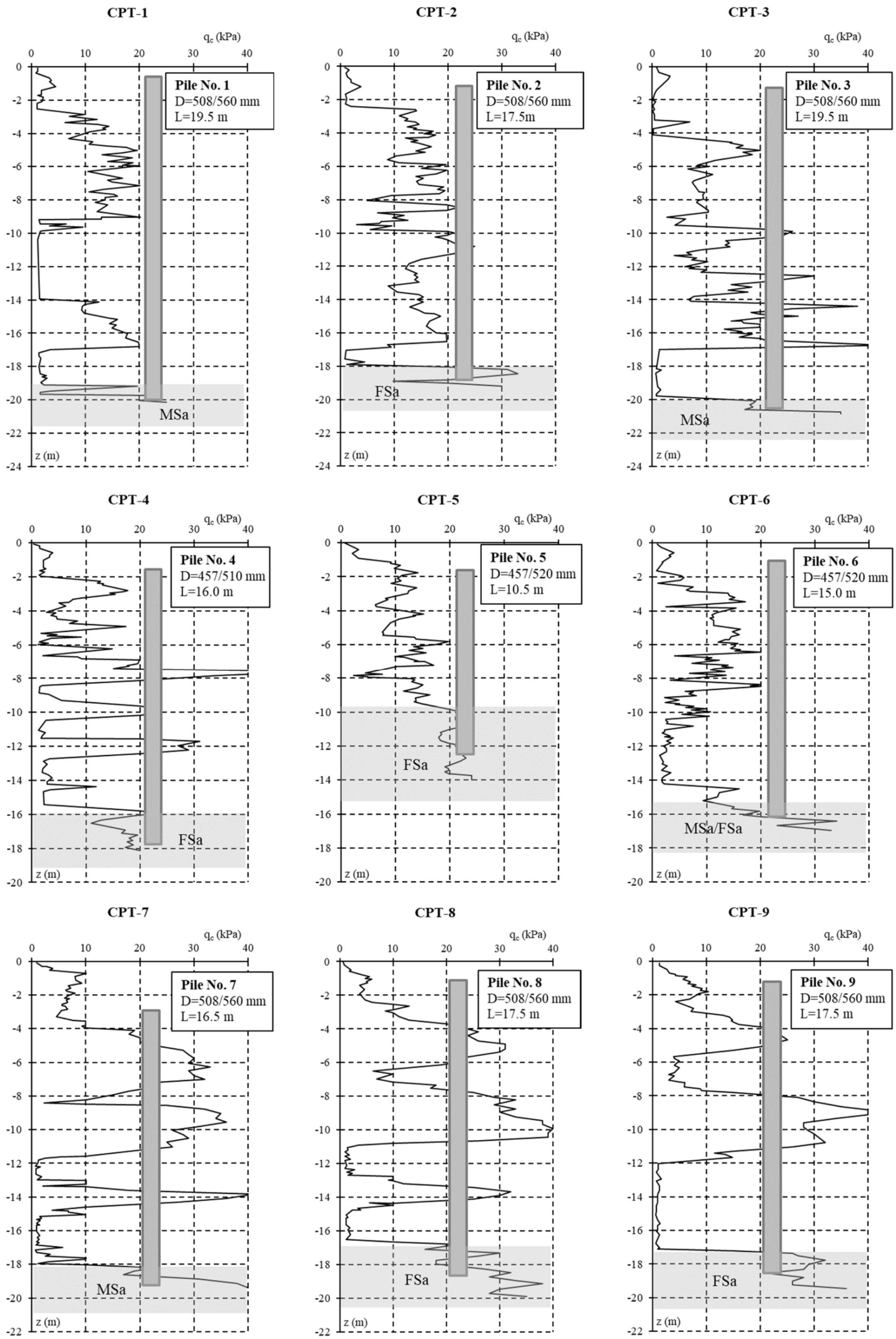


Figure 9: Diagrams of resistance q_c for example piles (Gdańsk: CPT-1 to CPT-3, Szczecin: CPT-4 to CPT-6, Grudziądz: CPT-7 to CPT-9).

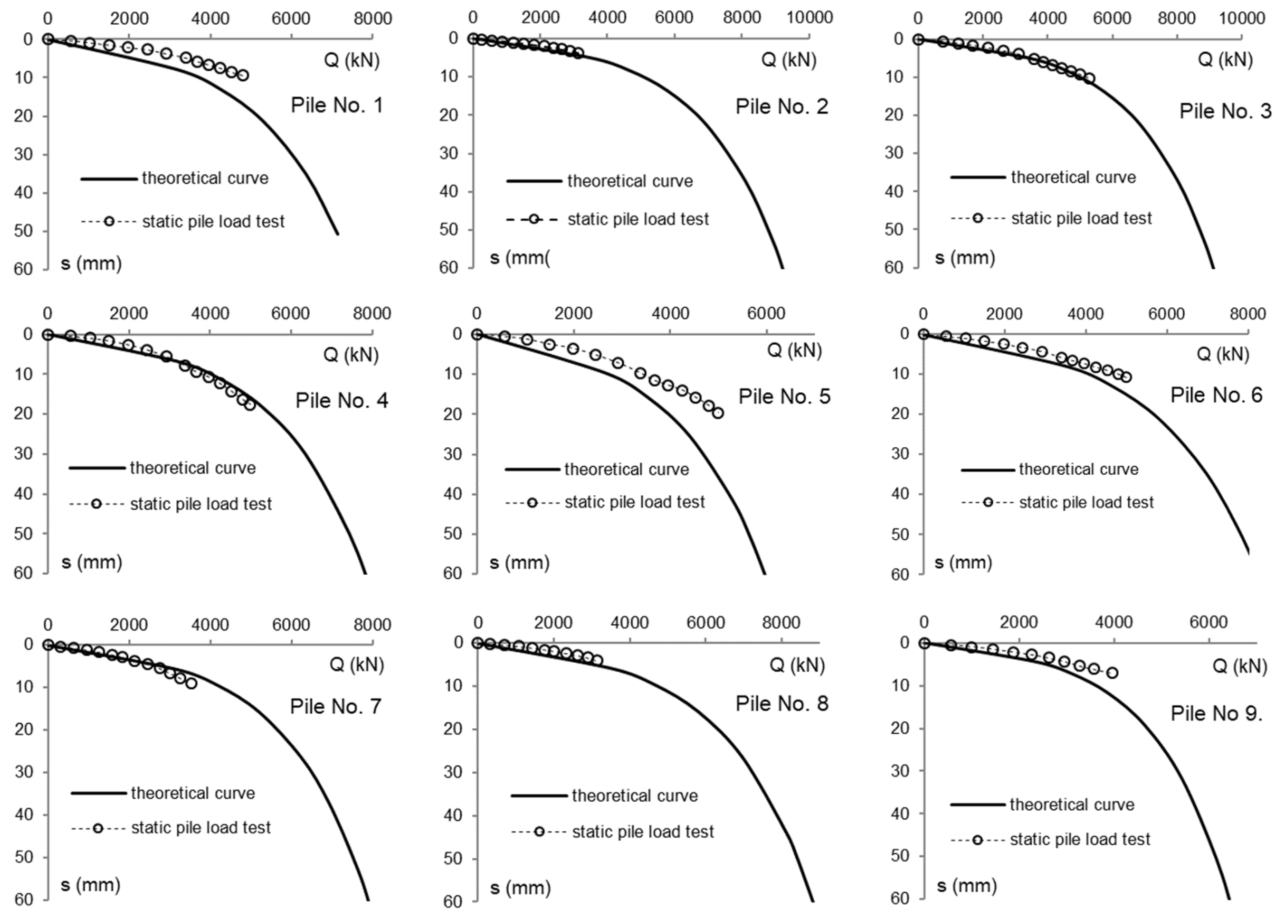


Figure 10: Comparison of theoretical curves with actual curves from static pile load tests.

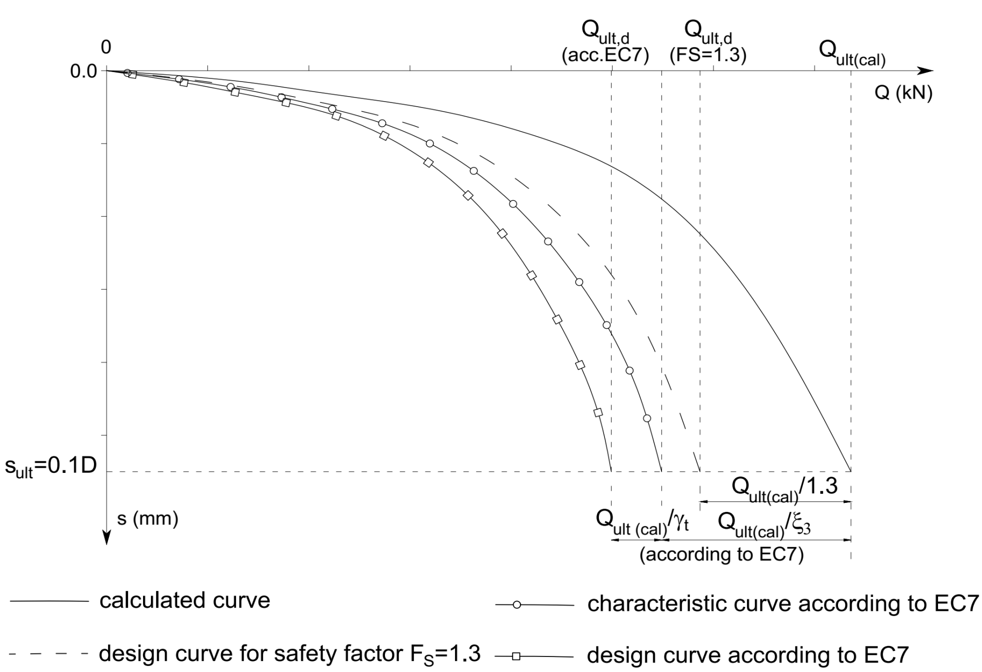


Figure 11: Concept for generation of safe design curve.

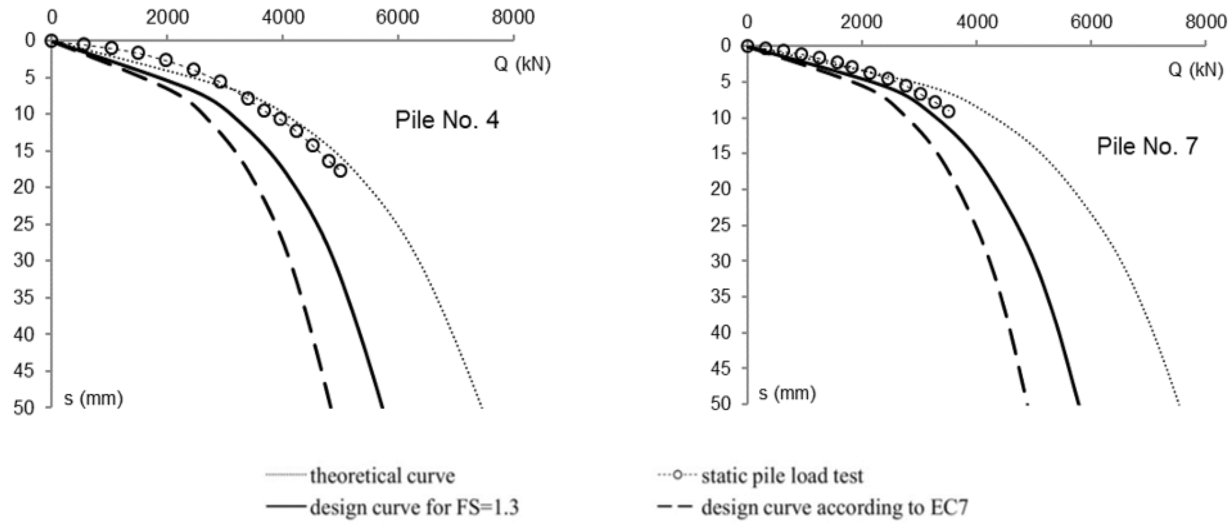


Figure 12: Comparison of design curves and curves from SPLT.

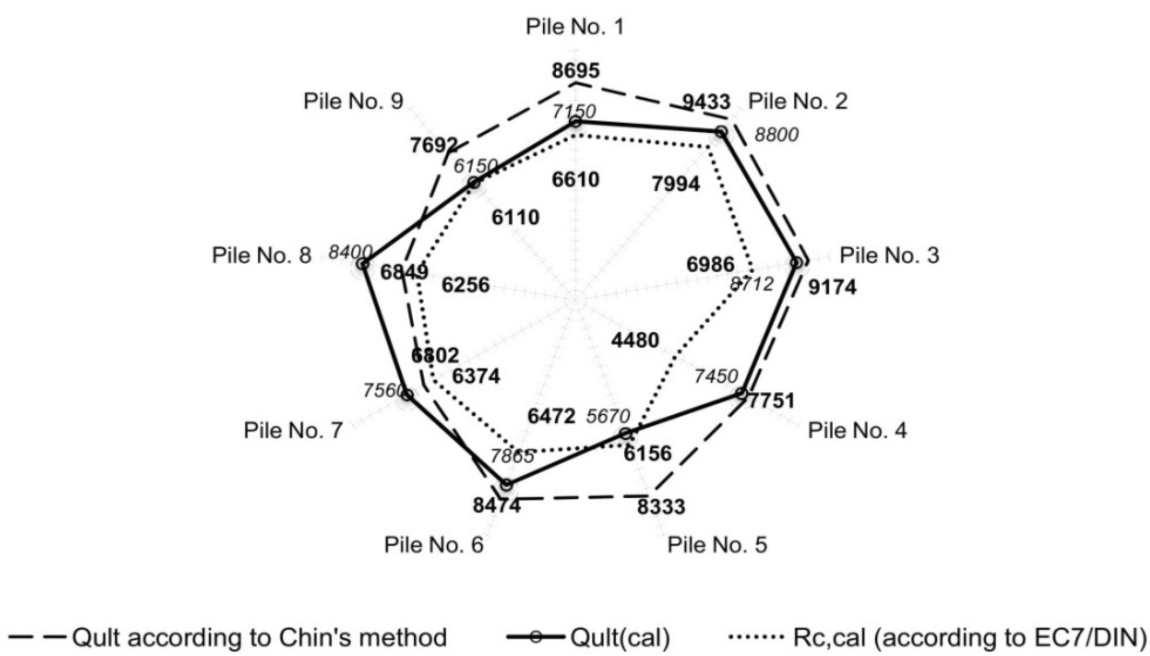


Figure 13: Values of calculated limit of load-bearing capacity.

Velloso's method and Penpile method. These relationships have been proven many times in many publications also by the author of this article [2, 14, 22-24].

It should also be borne in mind that the values obtained from the proposed method are more accurate than those obtained from the analytical method ($R_{c,d}$ (EC7/DIN)), proposed in the standard PN-EN 1997-2:2008 Eurocode 7: Geotechnical design. – Part 2: Ground investigation and testing, based on DIN1054 [20].

The graph presented in Fig. 14 proves that using a safety factor Fs = 1.3 gives a safe estimate of the load-bearing capacity.

Not only for the nine examples shown, but also for all 100+ cases considered in the analysis, all theoretical settlement curves run under the curves from the SPLT extrapolated using China's method. They are safe for design purposes. When using coefficients that are consistent with the DA2 approach, according to the current standard,

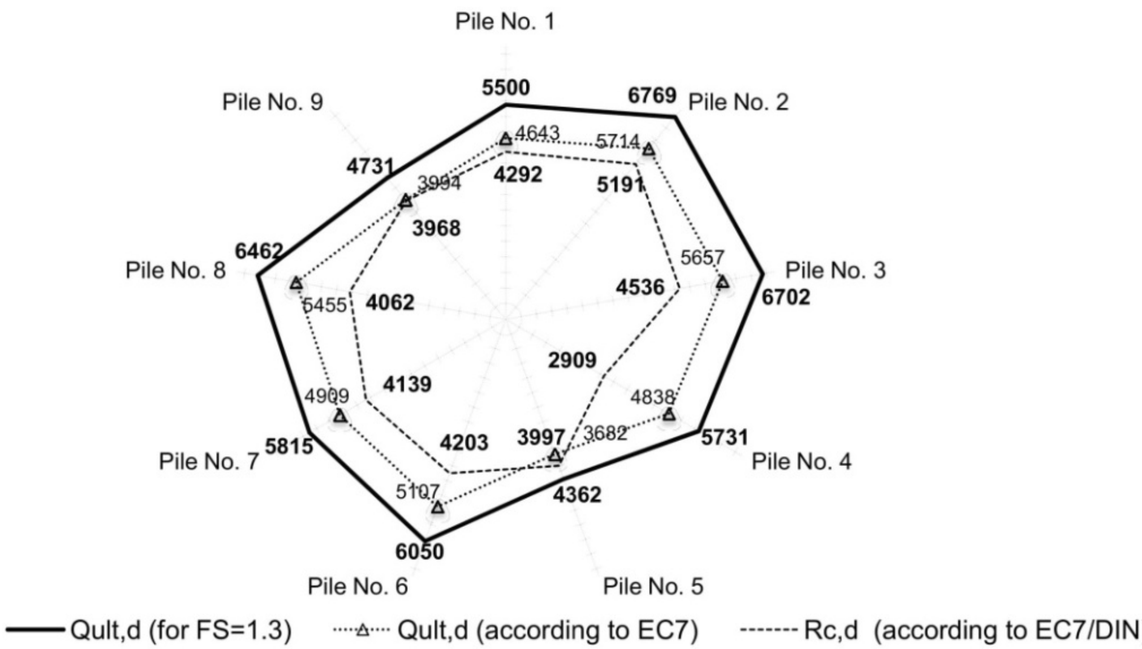
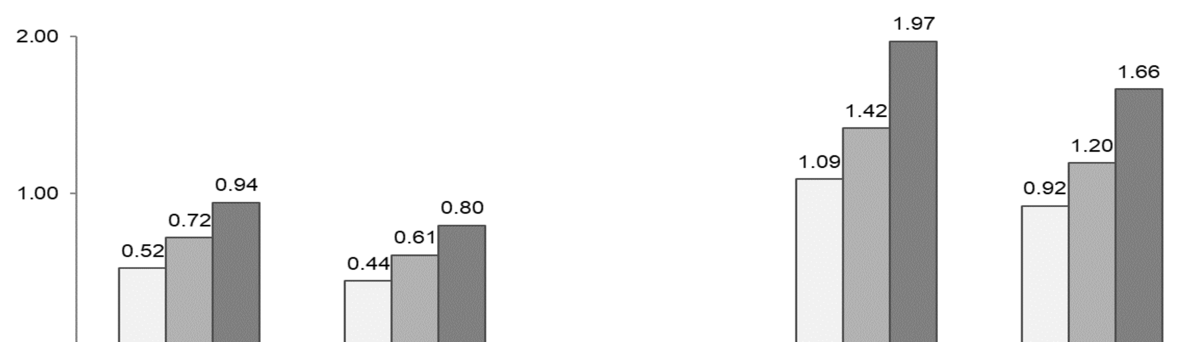


Figure 14: Comparison of design load-bearing capacity from diverse design methods.



	Qult,d (FS=1.3)/Qult (Chin)	Qult,d (EC7)/Qult (Chin)		Qult,d (FS=1.3)/Rc,d (EC7/DIN)	Qult,d (EC7)/Rc,d(EC7/DIN)
□ Min.	0.52	0.44		1.09	0.92
▨ Avg.	0.72	0.61		1.42	1.20
■ Max.	0.94	0.80		1.97	1.66

Figure 15: Summary of ultimate bearing capacity values from diverse design methods.

the reserve range increases. However, with respect to the results obtained from the calculation method contained in EC7/DIN1054, these values are higher. This allows Vibro piles to realise their true potential in load transfer.

The ultimate load-bearing capacity from the developed correlations is on average 28% lower than values from the Chin method, based on the actual Q–s curve. This assures sufficient reserve for design needs. However, if the set of coefficients from EC7 is used, it enables a very safe calculation of the load bearing with a reserve up to 39%. Despite such a significant reserve, the results are not overly cautious, which may be proven by differences as compared to the load-bearing capacity values obtained based on the standard method (Fig. 15).

5 Summary

The paper presents a proposal for a prediction method of Q–s curve course. The developed method belongs to a group of direct methods that make use of the results of CPT to determine theoretical load-bearing capacities for

Vibro piles with their base situated in non-cohesive soils. It has been defined on the basis of static load tests, and the necessary parameters for determination of full Q–s curve are cone resistance q_c and pile geometry. The defined correlations apply to piles whose base is located in fine and medium sands in moderately compacted, compacted and very compacted condition, that is, with a resistance value of qc > 13 kPa [25]. They can be successfully used to calculate Vibro piles with typical diameters ranging from 408 to 610 mm and of various lengths.

The analysis has shown that the obtained values of ultimate load bearing are similar to the results obtained in SPLT interpretation methods. Particular attention should be drawn to the possibility of more precise estimation of pile ultimate load-bearing capacities, as compared to EC7 and other direct methods based on the results of CPT probing. Comparisons were made with the values of ultimate bearing capacity, corresponding to 0.1D settlements, obtained from the method contained in EC7. This is because it is the method whose results are usually the highest and closest to the actual Vibro pile capacities. This is proven by numerous publications [2, 14, 22-24]. The differences between the results obtained from the different direct methods based on CPT are due to the adopted criterion of safe bearing capacity, characterised in detail by Gwizdała [9, 23]. Results of calculations performed based on popularly used analytical methods may be as follows:

– critical (indirect) bearing capacity for settlement within the range of 3%–5% of the pile diameter D [23];
– ultimate bearing capacity Q_{ult}, conforming to conventional settlements s_{ult} equalling to 10% of the pile diameter, as, for example, the methods presented in PN-EN 1997:2008, which have been based on the German standard DIN 1054 and the Dutch standard NEN 6743 [16].

Classical transformation functions, whose idea is to fit the settlement curve to the calculated ultimate bearing capacity, enable to plot the hyperbolic characteristics of settlement [9, 12-13]. The hyperbola does not represent the actual shape of the Q–s curve for Vibro piles. In the proposed new method of predicting the interaction of Vibro piles with the ground, it is possible to determine a two-phase course of the curve: a linear phase describing elastic interaction, characteristic for Vibro piles and confirmed by test pile loads, and a non-linear elastic–plastic phase.

An aspect that distinguishes the developed method is the possibility of determining the full pile load–settlement curve, based on the results of soil testing. If the full curve Q–s is available, it is possible to assume for design needs values of critical or ultimate bearing capacities or any indirect settlement values.

References

[1] M. J. Tomlinson, Pile design and construction practice. Taylor and Francis (1994).
[2] P. Więcławski, Methodology for estimating settlement of Vibro piles based on CPT. Politechnika Gdańsk (2016).
[3] G. Baldi, R. Bellotti, V.N. Ghionna, M. Jemiolkowski, D.C.F. Lo Presti, Modulus of sands from CPT and DMT. Proc. 12-th International Conference on Soil Mechanics and Foundation Engineering. Rio de Janerio. Balkema Rotterdam, Vol. 1 (1989).
[4] P. Bandini, R. Salgado, Methods of piles design based on CPT and SPT results. Balkema, Rotterdam (1998).
[5] C. P. Wroth, The Interpretation of in situ Soil Tests. Geotechnique, Vol. 34, No 4 (1984).
[6] A. Mandolini, Design of axially loaded piles – Italian practice. Proceeding International Seminar on Design of Axially Loaded Piles, European Practice, Brussels (1997).
[7] M. Bustamante, L. Gianeselli, Pile Bearing Capacity Prediction by Means of Static Penetration CPT. Proceedings of 2nd European Symposium on Penetration Testing, Amsterdam, Vol. 2 (1982).
[8] J. DeRuiter, F. L.Beringen, Pile Foundations for Large North Sea Structures. Marine Geotechnology, 3(3) (1979).
[9] K. Gwizdała, P. Więcławski, Polish experience in the assessment of pile bearing capacity and settlement of the pile foundation. Baltic Piling Days, Estonia, Talin (2012).
[10] G. G. Meyerhof, Bearing Capacity and Settlement of Pile Foundations. Journal of Geotechnical Engineering, ASCE, 102(3) (1976).
[11] R. Salgado, J. Lee, Pile Design on Cone Penetration Test Results. Final Report, FHWA/IN/JTRP-99/8 (1999).
[12] K. Gwizdała, Polish design methods for single axially loaded piles. Proceedings ERTC3 Seminar Design of Axially Loaded Piles. European Practice, 291-306, Brussels, Balkema (1997).
[13] A. Krasiński, Proposal for calculating the bearing capacity of screw displacement piles in non-cohesive soils based on CPT results. Studia Geotechnica et Mechanica, Vol. XXXIV, No. 4 (2012).
[14] K. Gwizdała, T. Brzozowski, P. Więcławski, Calculation aspects used in Eurocode 7 for pile foundation. From Research to Design in European Practice. Bratislava (2010).
[15] K. Gwizdała, M. Stęczniewski, Determination of the bearing capacity of pile foundations based on CPT test results. Studia Geotechnica et Mechanica No. 1-2/2007, Wrocław (2007).
[16] CEN (European Committee for Standardization), “Geotechnical design, part 1. General rules. *Eurocode 7*, Brussels (1997).
[17] E.M. Smed, P, Cundall. Elasto-plasto Strain Hardeninh Mohr-Coulomb Model-Derivation and Implementation, Aalborg, Denmark (2012).
[18] M. Strafield, P. Cundall, Towards a methodology for rock mechanics modelling International Journal of Rock Mechanics and Mining Science (1988).

[19] Wang Fan, M. Keer Leon, Numerical Simulation for Three Dimensional Elastic-Plastic Contact with Hardening BehaviorJ. Tribol 127(3) (2005).
[20] CEN (European Committee for Standardization), “Geotechnical design, part 2. Ground investigation and testing. *Eurocode 7*, Brussels (1997).
[21] F. K. Chin, Estimation of the ultimate load of piles from tests not carried to failure. Proceedings of the Second Southeast Asian Conference on Soil Engineering (1970).
[22] P. Więcławski, Evaluation of the interaction of Vibro piles with the soil on the basis of in-situ tests. Selected issues in construction and building materials and geotechnics. Bydgoszcz (2015).
[23] K. Gwizdała, P. Więcławski, Ultimate bearing capacity and interpretations of direct methods for displacement piles. Selected issues in construction and building materials and geotechnics. Bydgoszcz (2015).
[24] P. Więcławski, Limit load capacity of axially loaded Vibro piles based on a static load test and results of CPT tests. Scientific and technical conference for young scientists. on the occasion of the 100th anniversary of the Faculty of Civil Engineering of the Warsaw University of Technology, Warsaw (2015).
[25] M. Tarnawski, M. Ura, The comparison of CPT and DPSH tests results in non-cohesive soils. Marine Engineering and Geotechnics, No. 1/2012, Gdańsk (2012).