VILNIUS GEDIMINAS TECHNICAL UNIVERSITY

FACULTY OF ENVIROMENTAL ENGINEERING

DEPARTMENT OF ROADS

ANALYSIS OF BEARING CAPACITY ESTABLISHMENT METHODS FOR DRIVEN PILES

Master's degree Thesis

Author: Jurijs Kovzeļs, IKTfmfu – 20

Lecturer: Dr. sc. ing., professor Ainārs Paeglītis

VILNIUS 2022

VILNIUS GEDIMINAS TECHNICAL UNIVERSITY

Jurijs Kovzels 20203474
(Student's given name, family name, certificate number)

Faculty of Environmental Engineering
(Faculty)

Innovative Road and Bridge Engineering IKTfmfu-20
(Study programme, academic group no.)

DECLARATION OF AUTHORSHIP
IN THE COURSE PROJECT

20.12.2021.
(Date)

I declare that my Course Project entitled Analysis of Bearing Capacity Establishment Methods for Driven Piles ____ is entirely my own work. I have clearly signalled the presence of quoted or paraphrased material and referenced all sources.

I have acknowledged appropriately any assistance I have received by the following professionals/advisers: ____

The academic supervisor of my Course Project is Dr. sc. ing., professor Ainārs Paeglītis

No contribution of any other person was obtained, nor did I buy my Course Project.

(Signature)

Jurijs Kovzels
(Given name, family name)

VILNIUS GEDIMINAS TECHNICAL UNIVERSITY
FACULTY OF ENVIROMENTAL ENGINEERING
DEPARTMENT OF ROADS

RIGA TECHNICAL UNIVERSITY
FACULTY OF CIVIL ENGINEERING
INSTITUTE OF TRANSPORT INFRASTRUCTURE ENGINEERING
DEPARTMENT OF ROADS AND BRIDGES

APPOVED BY
Head of Department
(Signature)
AINARS PAEGLITIS
(Name, Surname)
03.01.2022
(Date)

Civil engineering study field

Innovative Road and Bridge Engineering study programme,

state code 6281EX002

OBJECTIVES FOR MASTER THESIS
..............No...............
Vilnius and Riga

For student Jurijs Kovzeļs
Master's Thesis title: Analysis of Bearing Capacity Establishment Methods for Driven Piles
Approved on 21 December 2021 by Dean's decree No.
(day, month, year)
The Final work has to be completed by 7 January 2022
(day, month, year)

THE OBJECTIVES:

1) compile different practical and theoretical pile bearing capacity establishment methods;
2) do analysis comparing established pile bearing capacity for piles using different pile driving formulas;
3) suggest rational and cost-efficient testing method (or combination of methods) for pile bearing capacity establishment.

Consultants of the Master's Thesis:
(Title, Name, Surname)

Academic Supervisor Dr. sc. ing., professor Ainārs Paeglītis
(Signature) (Title, Name, Surname)

Objectives accepted as a guidance for my Master's Thesis

(Student's signature)
Jurijs Kovzeļs
(Student's Name, Surname)
03.01.2022.
(Date)

Anotacija

Magistro darbo tema – Kaltinių polių laikomosios gebos nustatymo metodų analizė. Darbo tikslas – surinkti įvairius praktinius ir teorinius polių laikomosios galios nustatymo metodus, atlikti polių kalimo formulėmis pagrįstų metodų analizę ir pasiūlyti racionalų ir ekonomišką polių laikomosios galios nustatymo metodą.

Darbe pateikiamas polių tipų paaiškinimas pagal galiojančias Europos normas, atsižvelgiant į montavimo būdą ir konstrukcinę medžiagą. Skirtingi polių laikomosios galios nustatymo būdai buvo suskirstyti į netiesioginius ir tiesioginius. Netiesioginių laikomosios galios nustatymo metodų grupė yra pagrįsta skirtingais dirvožemio tyrimų duomenimis. Darbe aprašyti trys tiesioginės polių laikomosios galios nustatymo būdai. Pateikti metodai yra statinės apkrovos bandymas, dinaminės apkrovos bandymas ir greitosios apkrovos bandymas.

Praktinėje baigiamojo darbo dalyje nagrinėjami polių laikomosios galios nustatymo metodai, pagrįsti polių kalimo formulėmis – lygtimis, kurios nustato polių laikomąją galią, atsižvelgiant į nustatytą smūgio ir plaktuko smūgio energiją. Iš šešių plieninių vamzdinių polių matavimų duomenų, kuriuose buvo statinės apkrovos ir dinaminės apkrovos bandymas (polių kalimo formulė), buvo analizuojamos ir palygintos dvi polių kalimo formulės.

Darbą sudaro 89 puslapiai, kuriuos sudaro 6 skyriai, 33 paveikslai ir 29 lentelės.

Abstract

Master’s Thesis topic is Analysis of Bearing Capacity Establishment Methods for Driven Piles. The aim of the work is to compile different practical and theoretical pile bearing capacity establishment methods, perform analysis of methods based on pile driving formulas and suggest rational and cost-efficient method for pile bearing capacity establishment.

In the Thesis explanation of pile types regarding current European Norms is provided depending on installation method and construction material. Different pile bearing capacity establishment methods were divided in indirect and direct methods. Group of indirect bearing capacity establishment methods are based on different soil investigation data. Three direct pile bearing capacity establishment methods are described in the Thesis. Given methods are static load test, dynamic load test and rapid load test.

Practical part of the Thesis concentrates on analysis of pile bearing capacity establishment methods based on pile driving formulas – equations that establish pile bearing capacity considering set per blow and hammer impact energy. From data that included static load and dynamic load test (pile driving formula) measurements of six steel tubular piles two pile driving formulas were analysed and compared.

Thesis consists of 89 pages that includes 6 chapters, 33 figures and 29 tables.

Table of contents

List of figures

List of tables

Introduction

In our days piling is considered as most cost-efficient solution in deep foundation construction and it involves many modern-day solutions in every field, such as, modern day materials and construction equipment, but piling in construction practice comes from the times since man started to establish secure dwellings near water.

In the ancient history piles were used to establish settlements erected on timber pile foundation near the water or for shore and dock construction, which was a necessity for sea traders, such as, Phoenicians who did use wooden sheet piles.

Romans did use piles for bridge foundations, dams or/and other water retaining structures. Famous Trajan's Bridge over the river of Danube near todays Serbian-Romanian border, which was built in 105 AD with total length of 1135 meters, had 20 masonry pillars supported on timber piles.

Many great European cities are mostly built on piles, such as, Venice or Amsterdam. Venice that started to grow into city in early eight century and Amsterdam founded in 1275 both are cities built on water.

With technological progress not only new materials for piling but also piling equipment started to develop. Through times most used material for piling was timber and piles were driven in soil using rams that transformed animal or man effort into potential energy. After the invention of steam engine first steam engine powered impact hammers were introduced. Those hammers improved efficiency of construction and gave opportunity to drive piles into harder soils. Modern-day impact hammers have similar operating but different powering principle.

Moving away from timber as main structural material nowadays the choice of material depends on several factors, such as, length of the pile, loading and environment conditions. While timber is still capable material for different situations concrete and steel are most used materials in today's construction practice.

However, analysis of pile bearing capacity that includes evaluation of soil conditions and its mechanical properties is relatively new comparing to piling itself. Nowadays it is clear that pile bearing capacity depends on two factors – strength of structural element itself and resistance of the soil around the pile. This resistance must be enough to avoid excessive displacements, rotations and any other movements that makes superstructure unstable.

It is important to mention that structural strength of the pile itself could be determined easily, though soil resistance is affected by many factors and could be drastically different and change through time at the same construction site.

To establish pile bearing capacity (soil resistance) many different approaches are developed, each with own positive and negative sides, for example, results of pile static load testing are in site measured data that gives real soil-pile interaction behaviour, but execution of those tests take considerably more time than dynamic load testing or rapid load testing that can be performed in minutes comparing to hours.

Considering this, the aim of the master's thesis is to compare and evaluate different resistance establishment methods (both theoretical and practical) for piles which are subjected to compressive loads. Master's thesis experimental part will consist of different pile driving formula analysis (dynamic load testing) approach comparison.

The set goal will be achieved by performing the following tasks:

- compile different practical and theoretical pile bearing capacity establishment methods;
- do analysis comparing established pile bearing capacity for piles using different pile driving formulas;
- suggest rational and cost-efficient testing method (or combination of methods) for pile bearing capacity establishment.

1. Types of piles

1.1. Classification of piles

Piles can be classified by different criteria, for example, structural material, cross-section type, length, execution method etc. In this thesis suggested approach is to differentiate piles using European standards. According to those standards piles can be divided in:

- Displacement piles (executed according to EN 12699);
- Bored or Replacement piles (executed according to EN 1536);
- Micropiles (executed according to EN 14199);
- Sheet piling (executed according to EN 12063) [1].

1.2. Displacement piles

By definition given in EN 12699 displacement pile is pile which is installed in the ground without excavation or removal of material from the ground except for limiting heave, vibration, removal of obstructions or to assist penetration [2].

Displacement piles can bet divided in two groups – prefabricated or cast in place.

1.1.1. Prefabricated displacement piles

Those piles either have hollow (tubular) or solid cross-sections that are driven into the soil. Solid sections can be made from timber, concrete or steel. For the hollow (tubular) cross-sections concrete and steel (hot-rolled or welded) are used.

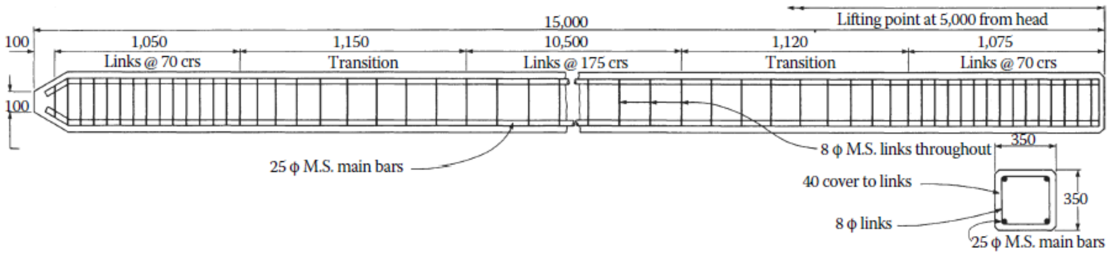

Fig.1.1. Typical solution for prefabricated reinforced concrete pile [1]

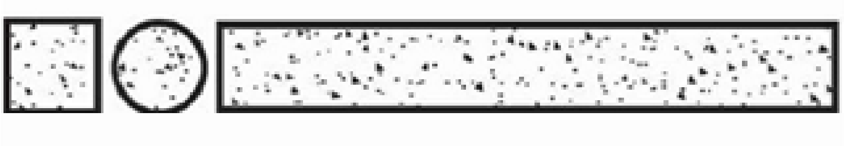

Fig.1.2. Typical cross-section for concrete pile [2]

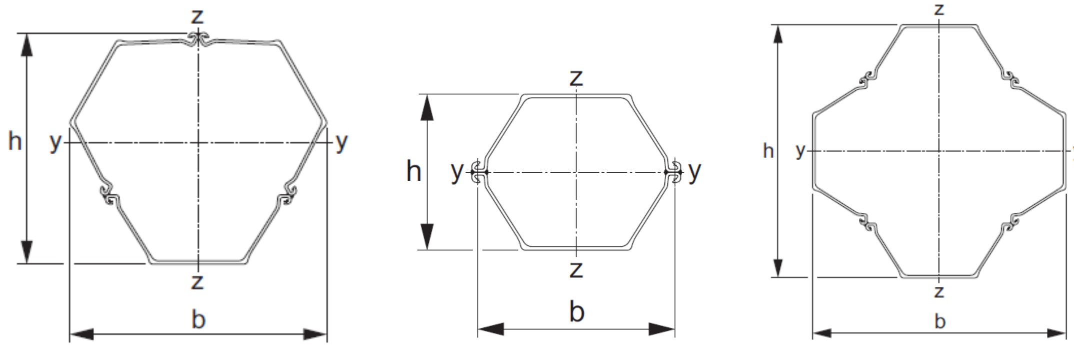

Fig.1.3. Typical cross-sections of prefabricated welded steel hollow piles [3]

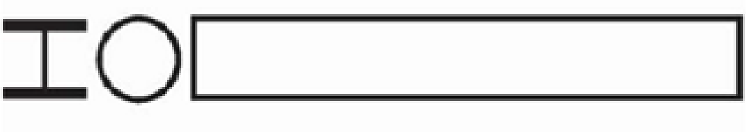

Fig.1.4. Typical cross-section of prefabricated hot-rolled steel piles [2]

Timber piles usually come with circular or square cross-section. Piles made of timber have several advantages. Timber has high strength to weight ratio; it is easy to handle and cut and in favourable conditions some timber species have indefinite life. Piles submerged in water are resistant to fungal decay, so it is good practice to build concrete cap just below lowest water level. As the main disadvantages that should be considered is limited length of timber piles, inhomogeneous material properties, deviations of pile itself and short lifespan above water level or poor decay protection [1].

Concrete piles usually come in square cross-section due to simplification of production. Main advantages of precast concrete piles are cost-efficiency, high quality of production and speed of installation (especially in easy driving conditions). High quality of production comparing to cast in place piles is reached because of factory production. Therefore, reinforcement location in cross-section is more precise and quality of concrete is guaranteed due to vibration through all length of the pile which is impossible for cast in place piles. European norm that sets requirements for precast concrete piles is EN 12794 [1].

Prefabricated concrete piles can be connected from several segments to increase total length of the pile which makes total length theoretically unlimited. Separate segments are connected using pile locks (see 1.5. picture) that should be tested according to EN 12794.

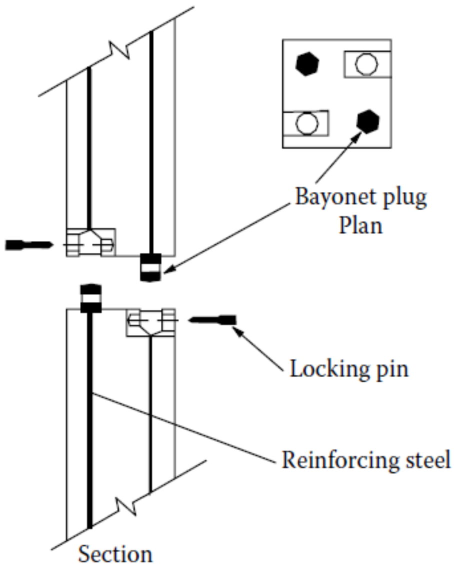

Fig.1.5. Locking pin joint for precast concrete pile [1]

Steel piles can be divided in to two groups – open ending and closed ending piles. Steel piles usually have circular H-section cross-section. Open ending piles have smaller bearing capacity due to smaller cross-sectional area at the pile base, however they easier to drive in hard soils comparing to closed end piles. Type of pile base does not affect the structural strength of pile itself. The main advantages are high yielding strength, robustness, easy handling, and ability to being driven into hard stratum. Steel piles are easy to cut and extend where needed.

1.1.2. Cast in place displacement piles

Cast in place piles are reinforced concrete piles casted into temporary or permanent steel casing. Installation of cast in place displacement piles starts with driving steel tube for desired depth. Steel tube could be driven with impact hammer into hard soils and screwed in situations where driving conditions are easy. Steel tube has sacrificial end cap that makes tube end closed. Sacrificial cap gives several advantages, such as, additional cross-sectional area at the pile end and, if performed correctly, it makes inside of the tube free from water which improves quality of the concrete pile [4].

After installation of steel tube reinforcement cage is lowered inside casing. After installation of reinforcement cage concrete is pumped from end of the pile. While concrete is pumped steel casing also is removed simultaneously.

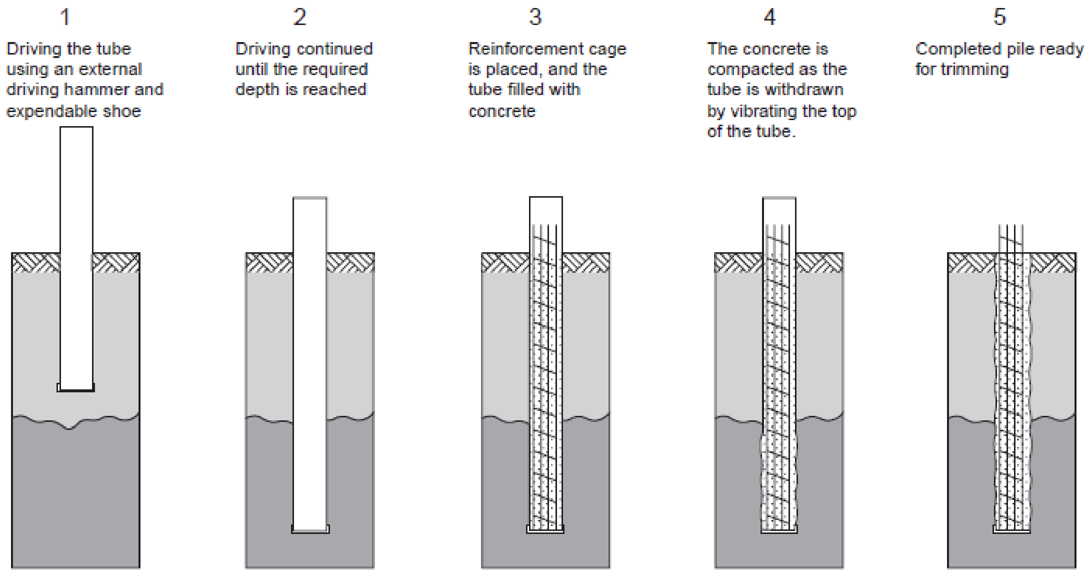

Fig.1.6. Installation sequence of cast in place concrete pile [4]

1.3. Bored or Replacement piles

By definition given in EN 1536 bored pile is a pile or barrette formed with or without a pile casing by excavating or boring a hole in the ground and filling with plain or reinforced concrete [5].

Bored piles are installed by first removing the soil by a drilling or percussion driving and then constructing the pile by placing concrete and/or reinforcement cage or other structural elements, for example, steel H-section [1].

Comparing to displacement piles bored piles has several advantages:

- depending on borehole execution technology bored piles can be placed in very stiff cohesive or very dense non-cohesive soils and even in rock, which is impossible with displacement piles;
- bored piles can be executed with larger size cross-sections (up to 3 meters in diameter).

As it is said in definition bored piles can be executed with or without casing. Most commonly used bored piles without casing are CFA (Continuous flight auger) piles because execution of those piles is fast and, in most cases, there is no necessity for additional measures during the execution. Common technologies for bored piles executed with casing are CCFA (Cased continuous flight auger) and Kelly type piles. Only difference between CFA and CCFA is that CCFA piles have temporary casing. Kelly piles from the other hand are completely different in execution. During borehole boring auger is located bellow separate casing elements. Casing elements are connected between each other during drilling, so length of the pile can reach more than 100 meters depending on geotechnical conditions. Significant difference during execution of CCFA and Kelly piles is in construction sequence. During construction of CCFA (also CFA) piles reinforcement cage is added to pile after concrete. Kelly piles are executed differently, during construction concrete comes after reinforcement, which also improves quality of end product because on practice it is significantly more complicated to install reinforcement cage in pile body, maintaining quality requirements.

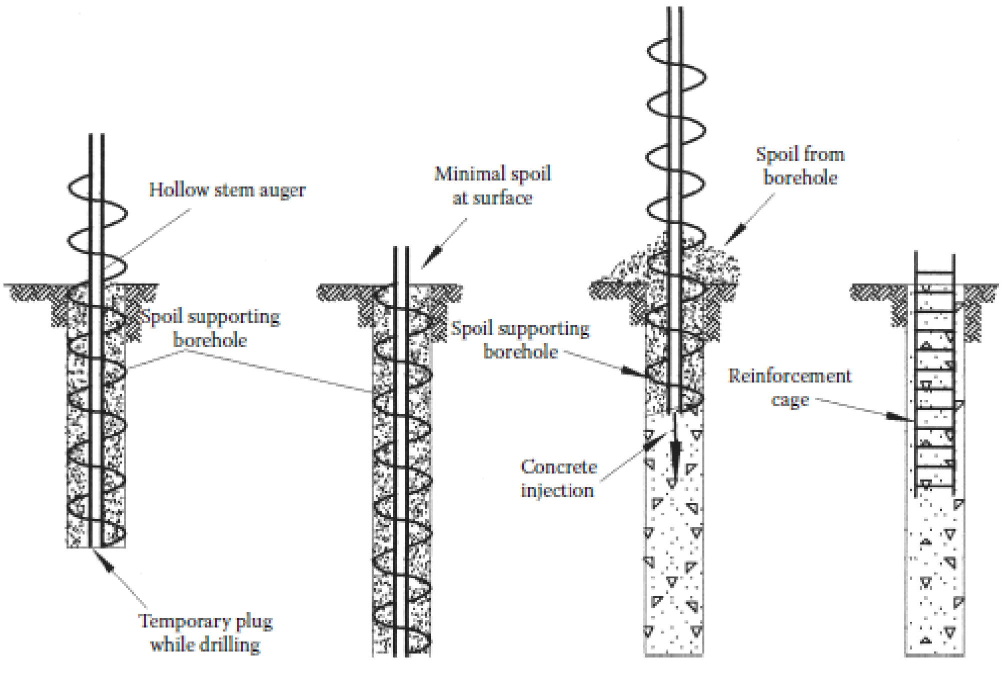

Fig.1.7. Installation sequence of CFA piles [1]

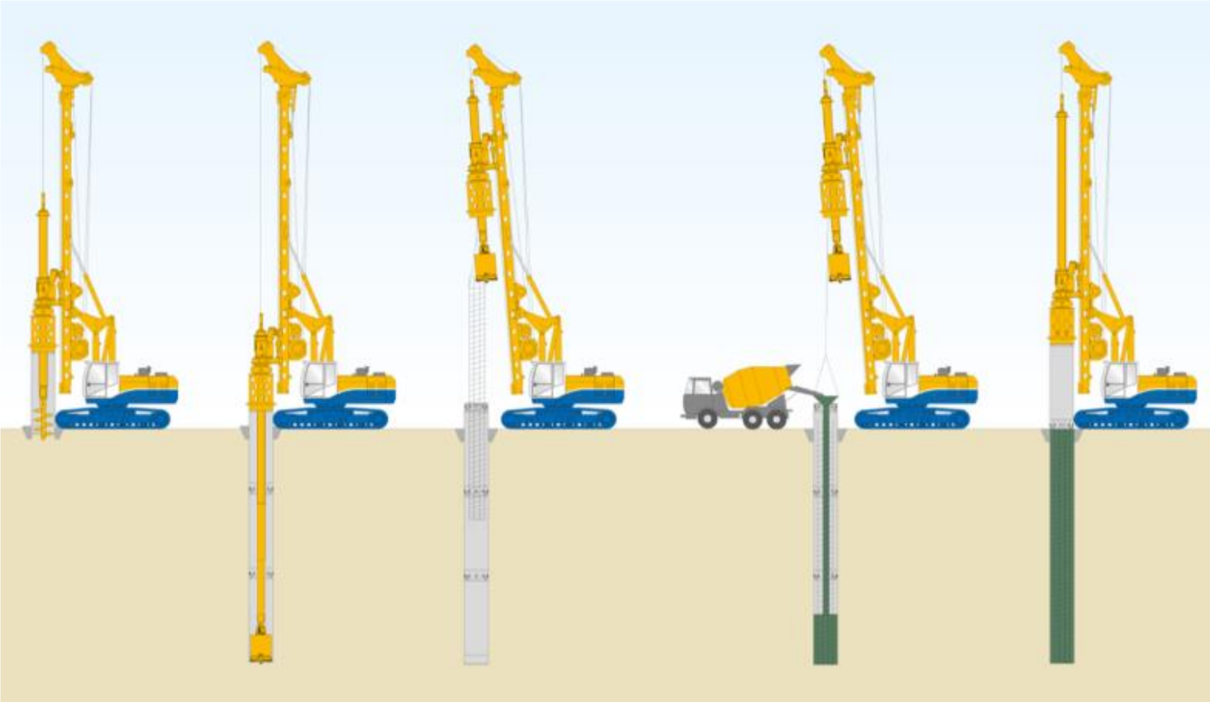

Fig.1.8. Installation sequence of Kelly piles [6]

Although several additional things should be considered while executing bored piles. First, decision towards usage of temporary casing should be made because performing borehole in unstable soils, such as, cohesive soils with high liquidity index, may affect quality of the borehole (straightness and cross-section area). As an alternative solution borehole could be strengthened with addition of bentonite suspension or polymer fluid to its shaft walls [5].

Second, even though execution of bored piles by drilling the borehole is vibration free and is considered safe pile installment approach in dense urban areas where buildings are close to each other, it is vitally important to take into account geotechnical conditions of construction site. Performing an uncased borehole through waterproof soil layers that keep constant pressure between waterproof layers and soil bellow will lead to pressure loss through the borehole, which will lead to changes of pile load bearing capacity of existing structures.

1.4. Micropiles

According to EN 14199 micropiles are bored piles with diameter less than 300 mm or displacement piles diameter or length of the shortest cross-section part less than 150 mm [7].

Micropiles can be executed not only as normal size displacement (both prefabricated and cast in place) or bored piles, but also to specific manufacturers technology that can include borehole drilling and pile grout body injection through one steel element that is used as load bearing element in completed micropile.

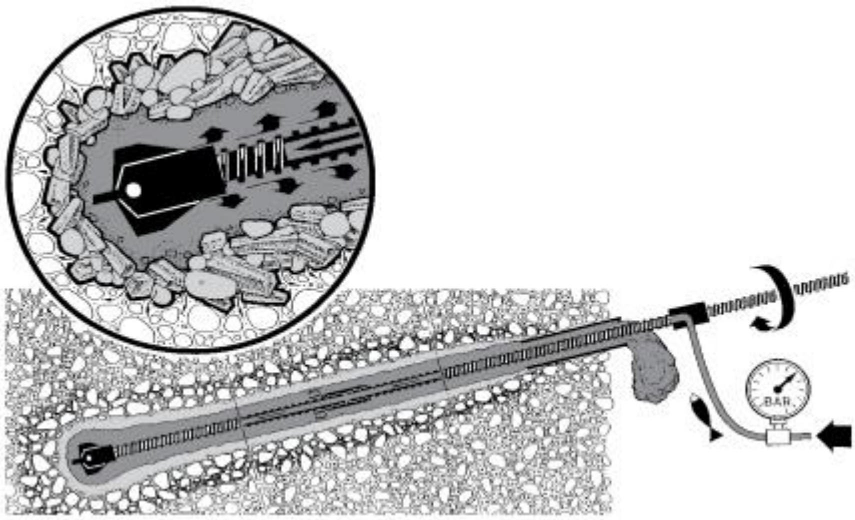

Fig.1.9. Ischebeck TITAN micropile grouting [8]

Micropiles are commonly used for foundations where compressive loads are small, as a tensile member to resist uplift or as an alternative for ground anchors. Micropiles executed according to EN 14490 “Soil nailing” are used for embankment reinforcement, protection from erosion or even as base points for rockfall protection nets near the roads [8].

This type of piles is widely used in narrow places where it is impossible to use normal piling equipment and for existing structures when it is necessary to improve stability, load bearing capacity or stop deformations.

According to EN 14199 micropiles are tested similarly to any other piles with exception regarding micropiles subjected to tension, which can be tested either as piles subjected to tension or as ground anchors according to EN 1537 [7].

1.5. Sheet piling

According to EN 12063 sheet pile is individual structural element of a sheet pile wall, consequently sheet pile wall is screen of individual sheet piles that form continuous walls. As structural material in sheet piling mainly steel and timber is used. Comparing to displacement, bored or micropiles sheet pile profiles are specifically designed to resist lateral loads and mainly used for retaining walls as elements subjected to bending.

Steel sheet piles can be divided in four shapes – Z-profiles, U-profiles, Straight web profiles (mainly used for cofferdams) and steel tubes. For structures with higher excavation depths combined wall systems including several elements is used.

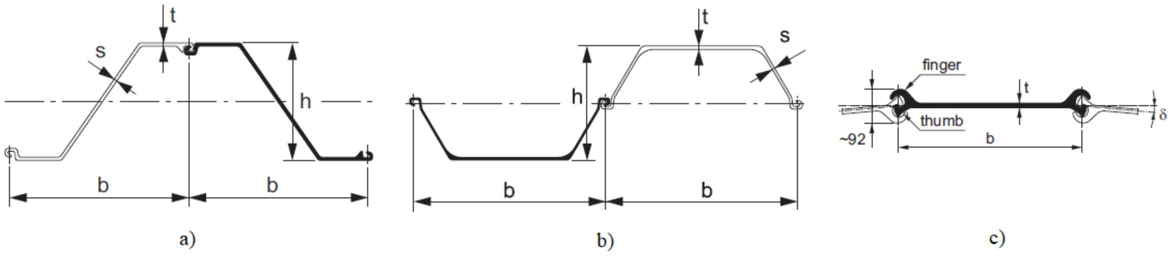

Fig.1.10. Standard sheet pile profiles a) Z-profile b) U-profile c) Straight web profile [3]

Combination of sheet pile elements for combined wall systems can be different but for all systems king and intermediary piles can be divided. King pile is main load bearing element with relatively high moment of inertia, such as, H beam or steel tube. Intermediary elements not only increase load bearing capacity but also provide continuity of the structure.

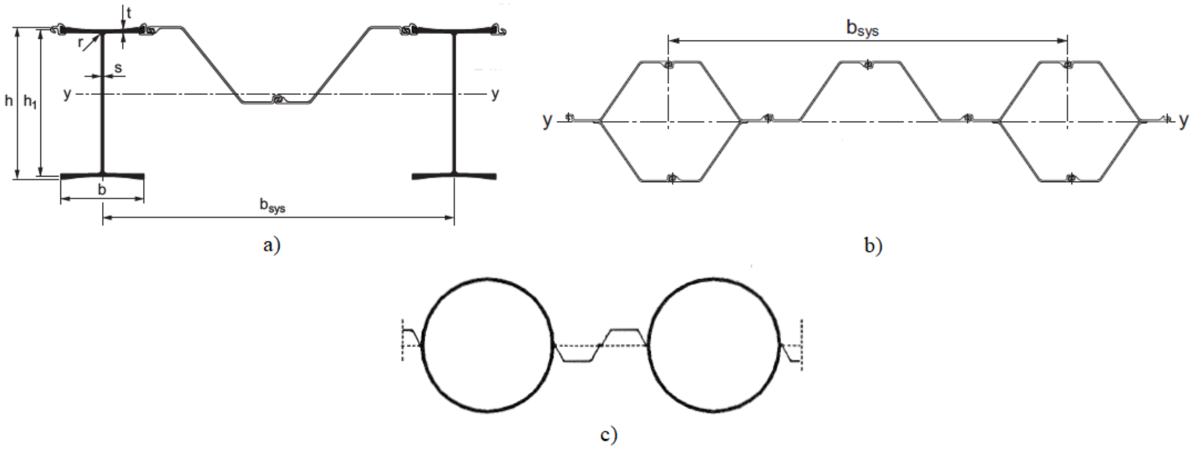

Fig.1.11. Standard combined walls a) H beam and Z-profile b) Z-profile (U-profile) box piles and Z-profile c) Steel tubes and U-profile (Z-profile) [3]

Timber sheet pile walls usually are formed from closely driven circular timber piles or timber planks. Less often processed timber piles of rectangular shape are used.

2. Indirect pile bearing capacity establishment methods

2.1. Introduction

In European Union design of pile foundations is performed according to Eurocode 7 (EN 1997-1) – Geotechnical design – Part 1: General rules. According to Eurocode 7 pile foundation design should be based on one of the following approaches:

- the results of static load tests;
- empirical or analytical calculation methods whose validity is proven by static load tests in comparable situations;
- the results of dynamic load tests whose validity is proven by static load tests in comparable situations;
- the observed performance of a comparable pile foundation, provided that this approach is supported by construction site geotechnical investigation [9].

Static load test is necessary operation in 3 out of 4 possible approaches, but it is important to note that last approach based on observations of existing foundation is rarely applied due to fact that too many factors should be similar to each other to state that foundations are comparable, for example, loading, pile execution technology, cross-section type and area, distance between piles, geotechnical conditions, pile length etc.

In this chapter several bearing capacity analysis methods will be reviewed based on most common geotechnical investigation results – Cone penetration test (CPT) results and general soil properties.

2.2. Establishment of pile bearing capacity

Pile bearing capacity should be divided in two parts – pile base bearing capacity and pile shaft bearing capacity. If pile end bearing capacity is significantly higher than shaft bearing capacity then pile is end-bearing pile, but if main contribution in total bearing capacity comes from shaft resistance, then pile is considered friction (or floating) pile [10].

Load bearing capacity is determined in accordance with the principles of Eurocodes – resistance is divided in characteristic and design values. According to EN 1997-1 pile bearing capacity without direct resistance testing should be established based on ground testing results. Pile design bearing capacity is given in Equation (2.1) [9].

$$\mathrm{R_{c;d}} = \mathrm{R_{b;d}} + \mathrm{R_{s;d}} \qquad (2.1)$$

where

$R_{c;d}$ – pile design bearing capacity (compressive resistance), kN

$R_{b;d}$ – pile design base bearing capacity, kN

$R_{s;d}$ – pile design shaft bearing capacity, kN.

Design values are established by Equation (2.2) and Equation (2.3).

$$R_{b;d} = \frac{R_{b;k}}{\gamma_b} \qquad (2.2)$$

where

$R_{b;d}$ – pile design base bearing capacity, kN

$R_{b;k}$ – pile characteristic base bearing capacity, kN

γ_b – partial factor for base resistance.

$$R_{s;d} = \frac{R_{s;k}}{\gamma_s} \qquad (2.3)$$

where

$R_{s;d}$ – pile shaft bearing capacity, kN

$R_{s;k}$ – pile characteristic shaft bearing capacity, kN

γ_s – partial factor for shaft resistance.

Characteristic values for base and shaft resistance can be established in two possible ways. First approach is expressed in Equation (2.4), and it depends on the number of profiles of tests.

$$R_{c;k} = \left(R_{b;k} + R_{s;k}\right) = \frac{R_{b;cal} + R_{s;cal}}{\xi} = \frac{R_{c;cal}}{\xi} = \min\left\{\frac{(R_{c;cal})_{mean}}{\xi_3}; \frac{(R_{c;cal})_{min}}{\xi_4}\right\} \qquad (2.4)$$

where

$R_{b;cal}$ – calculated pile base load bearing capacity, kN

$R_{s;cal}$ – calculated pile shaft bearing capacity, kN

$R_{c;cal}$ – calculated total pile bearing capacity, kN

$(R_{c;cal})_{mean}$ – mean calculated total pile bearing capacity, kN

$(R_{c;cal})_{min}$ – minimal calculated total pile bearing capacity, kN

ξ, ξ_3 and ξ_4 – correlation factors which depend on number of ground tests performed given in Table 2.1.

Table 2.1. Correlation factors ξ for characteristic resistance establishment from ground test results [9]

Number of tests	1	2	3	4	5	7	10
ξ_3	1.40	1.35	1.33	1.31	1.29	1.27	1.25
ξ_4	1.40	1.27	1.23	1.20	1.15	1.12	1.08

Given approach suggests analysis of pile resistance considering several points of ground testing. Characteristic value is established as minimal value between mean and minimal values corrected with correlation coefficients.

Other approach to establish characteristic base and shaft resistance values is suggested in Equation (2.5) and Equation (2.6) [9].

$$R_{b;k} = A_b q_{b;k} \qquad (2.5)$$

where

A_b – pile base cross-section area, m^2

$q_{b;k}$ – characteristic soil resistance at pile base, kN/m^2.

$$R_{s;k} = \sum_i A_{s;i}\, q_{s;i;k} \qquad (2.6)$$

where

$A_{s;i}$ – pile friction area in a separate soil layer, m^2

$q_{s;i;k}$ – characteristic soil friction resistance in a separate soil layer, kN/m^2.

Values $q_{b;k}$ and $q_{s;i;k}$ are obtained from different ground parameters depending on calculation method. Calculation methods used to determine these values are different and are chosen considering ground conditions, soil type and available information. Applicable calculation methods usually are fixed in National annex of EN 1997-1 for each country. It is important to mention that alternative methods can require additional safety factor that also should be fixed in National annex of Eurocode 7.

2.3. Pile bearing capacity calculation based on CPT results

2.3.1. Cone penetration testing

Cone penetration testing consists of advancing a specific size cylindrical rod with conical tip into the soil under constant rate of penetration. According to EN 1997-2 Cone penetration tests are divided in CPT, CPTU and CPTM tests. CPT and CPTU tests are executed according to EN ISO 22476-1. Difference between two testing methods is that in addition to all main parameters, which must be determined during testing, CPTU test results contain pore water pressure measurements and resulting q_c values are corrected for pore water pressure effects. CPTM test is executed according to EN ISO 22476-12 and unlike CPT or CPTU testing it is performed manually [11].

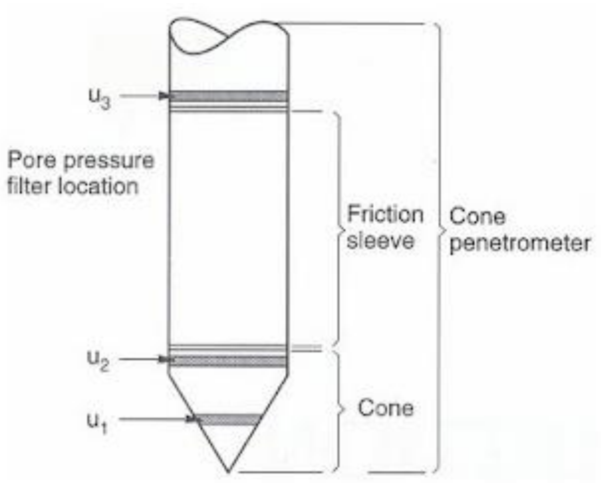

Fig.2.1. Cone penetrometer [12]

During testing friction cone penetrometer measures two forces – total tip resistance (q_c), which is soil resistance at the tip, and sleeve friction (f_s), which is friction force between friction sleeve and surrounding soil. Ratio between those two parameters is called friction ratio (R_f) and it is expressed in percent [13].

Understanding values from CPT results can give picture about soil type and can be interpreted into other physical or mechanical properties of the soil. Fine soils (clays) have low tip resistance, high sleeve friction and friction ratio values compared to coarse (sands) soils, whose tip resistance is high, but sleeve friction and friction ratio is low. Moreover, coarse soils have low pore pressure, but fine soils in opposite have high pore pressure [14].

Not only it is possible to determine soil type by CPT results, but also possible to establish other parameters frequently used in pile resistance calculations, such as, soil unit weight, angle of shearing resistance (friction angle), density index or undrained shear strength of fine soil.

Soil unit weight can be established with Equation (2.7) or Equation (2.8) if soil specific gravity values are obtained in laboratory testing [15].

$$\frac{\gamma}{\gamma_w} = 0.27 * [\log(R_f)] + 0.36 * \left[\log\left(\frac{q_c}{p_a}\right)\right] + 1.236 \qquad (2.7)$$

where

γ – unit weight of soil, kN/m^3

γ_w – unit weight of water, kN/m^3

R_f – friction ratio, %

q_c – cone tip resistance, N/mm^2

p_a – atmospheric pressure, N/mm^2.

$$\frac{\gamma}{\gamma_w} = \left[0.27 * [\log(R_f)] + 0.36 * \left[\log\left(\frac{q_c}{p_a}\right)\right] + 1.236\right] * \frac{G_s}{2.65} \qquad (2.8)$$

where

G_s – soil specific gravity, g/cm^3.

To establish density index and angle of shearing resistance for non-cohesive soils Table 2.2. can be used.

Table 2.2. Density index and angle of shearing resistance based on CPT results for non-cohesive soils [16]

Cone tip resistance q_c, MPa	Density index	Angle of shearing resistance φ, °
0.0 – 2.5	Very loose	29 – 32
2.5 – 5.0	Loose	32 – 35
5.0 – 10.0	Medium dense	35 – 37
10.0 – 20.0	Dense	37 – 40
> 20.0	Very dense	40 – 42

Similar table can be found in EN 1997-2, and it could be added that values of angle of shearing resistance are given for sands. For silty soils angle of shearing resistance should be reduced by 3°, for gravels 2° should be added [11].

Undrained shear strength of fine (cohesive) soil can be established directly from CPT or CPTU results depending on which one was performed by Equation (2.9) for CPT and Equation (2.10) CPTU [11].

$$c_u = \frac{q_c - \sigma_{v0}}{N_k} \qquad (2.9)$$

where

c_u – undrained shear strength of fine soil, kN/m^2

q_c – cone tip resistance from CPT results, kN/m^2

σ_{v0} – initial total vertical overburden stress at considered depth, kN/m^2

N_k – empirical correlation coefficient.

$$c_u = \frac{q_t - \sigma_{v0}}{N_{kt}} \qquad (2.10)$$

where

q_t – cone tip resistance from CPTU results (corrected for pore water effects), kN/m^2

N_{kt} – empirical correlation coefficient that depends on cone geometry, OCR and lies between 10 and 20 [17].

2.3.2. Pile bearing capacity calculation according to EN 1997-2

Annex D of EN 1997-2 is dedicated to variety of things in context of geotechnical design using CPT results. Regarding calculation of pile bearing capacity annex D.7 should be used.

The maximum compressive resistance of a pile according to annex D.7 is given in Equation (2.11) [11].

$$F_{max} = F_{max;base} + F_{max;shaft} \qquad (2.11)$$

where

F_{max} – maximum compressive resistance of the pile, kN

$F_{max;base}$ – maximum base resistance of the pile, kN

$F_{max;shaft}$ – maximum shaft resistance of the pile, kN.

To comply with Eurocode principles described in section 2.2. given Equation (2.11) should be rewritten as Equation (2.12).

$$R_{c;cal} = R_{b;cal} + R_{s;cal} \qquad (2.12)$$

Values $R_{b;cal}$ and $R_{s;cal}$ are calculated from Equation (2.13) and Equation (2.14).

$$R_{b;cal} = F_{max;base} = A_b p_{max;base} \qquad (2.13)$$

where

$p_{max;base}$ – maximum unit base resistance calculated by Equation (2.15), N/mm^2.

$$R_{s;cal} = F_{max;shaft} = u \int_0^{\Delta L} p_{max;shaft;z} dz \qquad (2.14)$$

where

u – perimeter of pile shaft, m

ΔL – the distance from the base of the pile to the bottom of the first soil layer with q_c value bellow 2 MPa and this value should be less than length of enlarged part of pile cross-section (if applied), m

$p_{max;shaft;z}$ – maximum unit shaft resistance at depth z, kN/m^2, calculated by Equation (2.22)

z – the depth or vertical direction, m.

$$p_{max;base} = 0.5\alpha_p\beta s\left(\frac{q_{c;1;mean} + q_{c;2;mean}}{2} + q_{c;3;mean}\right) \leq 15\ \mathrm{N/mm^2} \qquad (2.15)$$

where

α_p – coefficient depending on pile type given in Table 2.3.

β – coefficient depending on pile base shape determined by Fig. 2.2.

s – coefficient depending on pile cross-section shape determined by Equation (2.16)

$q_{c;1;mean}$ – the mean of the q_c values over the depth from pile base to level critical depth d_{crit} established by Equation (2.17)

$q_{c;2;mean}$ – the mean of the lowest q_c values over the depth from of critical depth going upwards to pile base established by Equation (2.18)

$q_{c;3;mean}$ – the mean of the q_c values over the depth from pile base going upwards to a level of $8D_{eq}$ established by Equation (2.19)

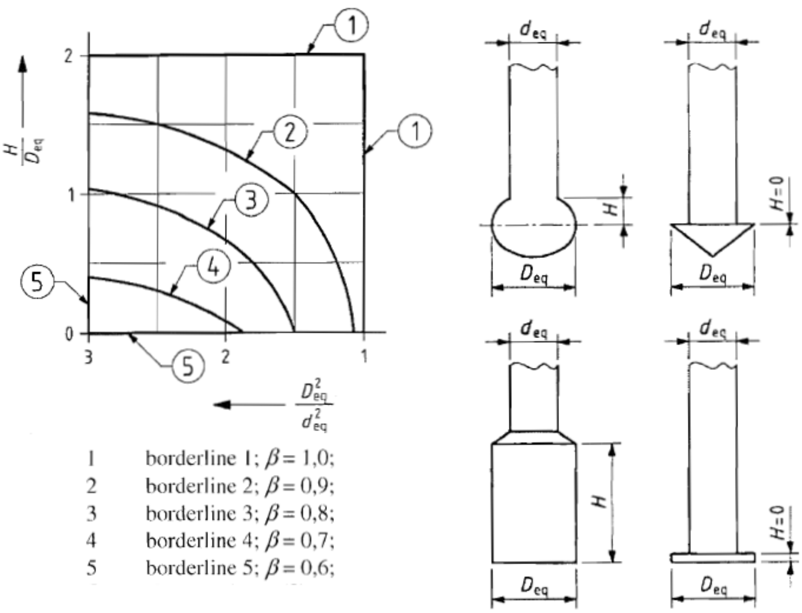

Fig. 2.2. Pile base shape factor [11]

$$s = \left(1 + \frac{\sin\varphi}{r}\right) / (1 + \sin\varphi) \qquad (2.16)$$

where

φ – angle of soil shearing resistance, °

r – ratio h/b (value is equal to 1 for square or circular cross-sections)

h – the larger side of the rectangular pile cross-section, m

b - the smaller side of the rectangular pile cross-section, m

$$q_{c;1;mean} = \frac{1}{d_{crit}} \int_{0}^{d_{crit}} q_{c;z;1} dz \qquad (2.17)$$

where

d_{crit} – critical depth calculated by Equation (2.20), m

$q_{c;z;1}$ – q_c value over the depth z, N/mm^2.

$$q_{c;2;mean} = \frac{1}{d_{crit}} \int_{d_{crit}}^{0} q_{c;z;2} dz \qquad (2.18)$$

where

$q_{c;z;2}$ – minimal q_c values over the depth z, N/mm^2.

$$q_{c;3;mean} = \frac{1}{8D_{eq}} \int_{0}^{-8D_{eq}} q_{c;z;3} dz \qquad (2.19)$$

where

D_{eq} – Equivalent pile diameter calculated by Equation (2.21), m

$q_{c;z;3}$ – mean q_c values over the depth z, N/mm^2.

This value cannot exceed 2 N/mm^2 for several bored type piles (CFA), unless CPT is performed after pile installation at distance closer than 1 m to the pile.

$$d_{crit} = 0.7D_{eq} < d_{crit} < 4D_{eq} \qquad (2.20)$$

$$D_{eq} = \sqrt{\frac{4A_p}{\pi}} \qquad (2.21)$$

where

A_p – pile cross-section area, m^2.

Explanation on establishing $q_{c;z}$ values over the depth of CPT results is given in Fig.2.3.

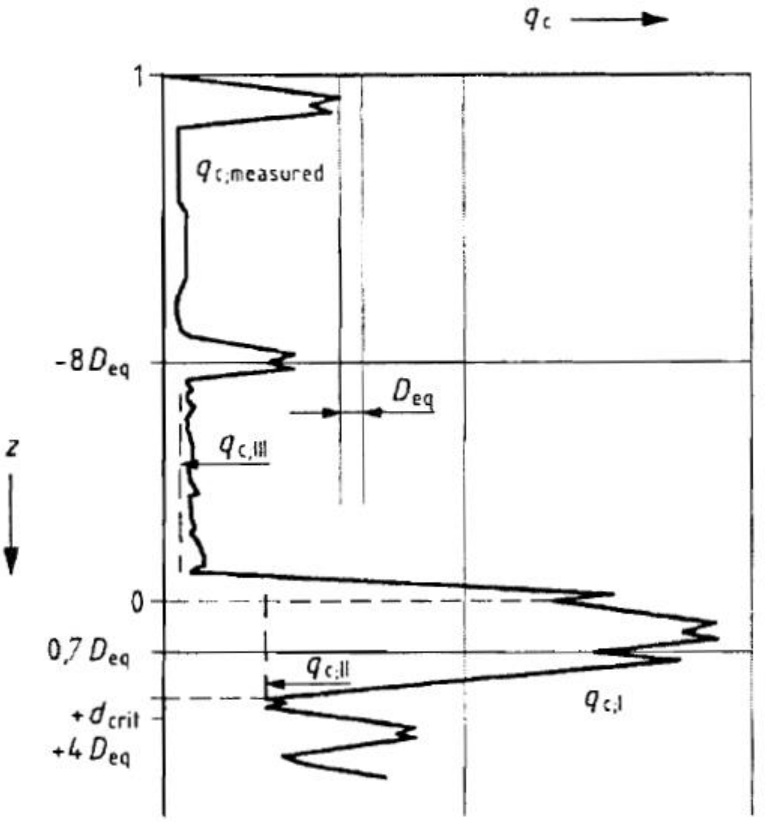

Fig.2.3. Explanation of $q_{c;z}$ values over CPT results [11]

The maximum shaft resistance is established by Equation (2.22) and it is derived from cone tip resistance.

$$p_{max;shaft;z} = \alpha_s q_{c;z} \qquad (2.22)$$

where

α_s – factor depending on soil and pile type given in Table 2.3. and Table 2.4.

$q_{c;z}$ – cone tip resistance value at depth z, N/mm^2.

Explanation on establishing $q_{c;z}$ values over the depth of CPT results is given in Fig.2.2.

Additionally, $q_{c;z}$ values over the interval with length more than 1 m cannot exceed 15 N/mm^2 if $q_{c;z}$ values over the interval are higher than 12 N/mm^2. If given interval is shorter than 1 m, then $q_{c;z}$ values should not exceed 12 N/mm^2 [11].

Table 2.3. Factor α_p values and factor α_s values for non-cohesive soils (sands and gravels) [11]

Pile type	α_p	α_s (sand)	α_s (coarse sand)	α_s (gravel)
Soi displacement piles with diameter > 150 mm				
Driven prefabricated piles	1.0	0.010	0.0075	0.005
Cased with closed end cast in place (with reclaimed casing)	1.0	0.012	0.009	0.006
Soil replacement piles with diameter > 150 mm				
Flight auger piles	0.8	0.006	0.0045	0.003
Bored piles (with drilling fluid)	0.6	0.005	0.00375	0.0025

Table 2.4. Factor α_s values for cohesive soils (silts and clays) [11]

Soil type	q_c, N/mm^2	α_s
Clay	> 3	< 0.030
Clay	< 3	< 0.020
Silt	-	< 0.025
Peat	-	0

It should be mentioned that in case of soil overconsolidation q_c values should be reduced, which can severely affect pile bearing capacity. However, calculation approach given in EN 1997-2 does not provide explanation on how over consolidation affects q_c values.

2.3.3. Pile bearing capacity calculation according to NEN 6743

NEN 6743 is Netherlands national code issued by the Royal Netherlands Standardization institute and calculation approach used in EN 1997-2 is based (with several differences) on this specific construction code.

According to this method same equations as for calculation according to EN 1997-2 are used. Differences between two methods appear in coefficients α_p and α_s. NEN 6743 provides more detailed differentiation of pile types and provides different approach in establishing α_s values. Also, this code gives explanation on overconsolidation effects on pile bearing capacity and q_c reduction depending on OCR.

Values of factor α_p according to NEN 6743 are given in Table 2.5.

Table 2.5. Factor α_p values [18]

Pile type	α_p
Driven prefabricated piles	1.0
Franki piles	1.0
Driven timber piles	1.0
Vibrating piles	1.0
Cast in place screw piles	0.9
Prefabricated screw piles	0.8
Cast in place screw piles with additional grouting	0.9
Prefabricated screw piles with additional grouting	0.8
Steel tubular piles	1.0
CFA piles	0.8
Bored piles or piles sheeted by bentonite suspense	0.5
Bored piles with steel casing	0.5

Values α_s in this approach are established in different way. In EN 1997-2 given factor for silt is constant but for clay it varies depending on q_c value. NEN 6743 includes new parameter – relative depth. Relative depth equals z/d_{eq} (depth of the soil layer divided by equal pile shaft diameter). Values of factor α_s according to NEN 6743 are given in Table 2.6.

Table 2.6. Factor α_s values [18]

Soil type	q_c, N/mm^2	z/d_{eq}	α_s
Clay/silt	≤ 1	$5 < z/d_{eq} < 20$	0.025
Clay/silt	≥ 1	$z/d_{eq} > 20$	0.055
Clay/silt	> 1	-	0.035
Peat	-	-	0

Influence of overconsolidation is taken into account by reducing maximum unit base resistance ($p_{max;base}$) by coefficient depending on OCR according to the Table 2.7.

Table 2.7. Influence of OCR on $p_{max;base}$ [18]

OCR	$p_{max;base}$
≤ 2	$p_{max;base}$
$2 < OCR \leq 4$	$0.67p_{max;base}$
$OVR > 4$	$0.5p_{max;base}$

2.4. Pile bearing capacity calculation based on general soil properties

Given alternative indirect bearing capacity establishment method is based on empirical values established for disperse (cohesive and con-cohesive) soils depending on parameters, such as, soil type, density index (I_D), particle size, liquidity (I_L) and plasticity (I_P) indexes, depth of the soil layer bellow surface and soil shearing resistance (inner friction) angle.

This approach is given in Latvian construction norm LBN 207-15 and national annex for EN 1997-1 allows to use this approach as an alternative calculation method when no CPT data is available. Model correction factor for this method according to national annex is 1.25 [9].

Values $q_{b;k}$ used in Equation (2.5) and $q_{s;i;k}$ used in Equation (2.6) are established differently for piles installed according to EN 12699 (Displacement piles) and EN 1536 (Bored piles).

Calculations by this method require to classify sandy soils by their granulometry according to Table 2.8.

Table 2.8. Sandy soil type depending on particle size [19]

Soil type	Requirement
gravely sand	2 mm particles exceed 25% of soil mass
coarse sand	0.5 mm particles exceed 50% of soil mass
medium coarse sand	0.25 mm particles exceed 50% of soil mass
fine sand	0.1 mm particles exceed 75% of soil mass
silty sand	0.1 mm particles do not exceed 75% of soil mass

These values are established according to Equation (2.23) and Equation (2.24) [19].

$$q_{b;k} = q_b \gamma_{cb} \gamma_c \qquad (2.23)$$

where

q_b – pile base unit resistance established according to Table 2.9. for displacement piles and Table 2.10. or Equation (2.25) for bored piles

γ_{cb} – coefficient depending on pile installation method and ground conditions established according to Table 2.11.

γ_c – load direction coefficient established according to Table 2.12.

$$q_{s;i;k} = q_{s;i} \gamma_{cs} \gamma_c \qquad (2.24)$$

$q_{s;i}$ – pile shaft unit resistance established according to Table 2.13.

γ_{cs} – coefficient depending on pile installation method and ground conditions established according to Table 2.11.

Table 2.9. Pile base unit resistance q_b for displacement piles [19]

<table>
<tr><td rowspan="5">Pile base depth, m</td><td colspan="7">Pile base unite resistance $q_{b,}$ kN/m^2</td></tr>
<tr><td colspan="7">$0.33 < I_D < 0.67$</td></tr>
<tr><td>Gravely sand</td><td>Coarse sand</td><td>-</td><td>Medium coarse sand</td><td>Fine sand</td><td>Silty sand</td><td>-</td></tr>
<tr><td colspan="7">Clayey soil ($I_P > 0.01$) with I_L</td></tr>
<tr><td>0.0</td><td>0.1</td><td>0.2</td><td>0.3</td><td>0.4</td><td>0.5</td><td>0.6</td></tr>
<tr><td>3</td><td>7500</td><td><u>6600</u>
4000</td><td>3000</td><td><u>3100</u>
2000</td><td><u>2000</u>
1200</td><td>1100</td><td>600</td></tr>
<tr><td>4</td><td>8300</td><td><u>6800</u>
5100</td><td>3800</td><td><u>3200</u>
2500</td><td><u>2100</u>
1600</td><td>1250</td><td>700</td></tr>
<tr><td>5</td><td>8800</td><td><u>7000</u>
6200</td><td>4000</td><td><u>3400</u>
2800</td><td><u>2200</u>
2000</td><td>1300</td><td>800</td></tr>
<tr><td>7</td><td>9700</td><td><u>7300</u>
6900</td><td>4300</td><td><u>3700</u>
3300</td><td><u>2400</u>
2200</td><td>1400</td><td>850</td></tr>
<tr><td>10</td><td>10500</td><td><u>7700</u>
7300</td><td>5000</td><td><u>4000</u>
3500</td><td><u>2600</u>
2400</td><td>1500</td><td>900</td></tr>
<tr><td>15</td><td>11700</td><td><u>8200</u>
7500</td><td>5600</td><td><u>4400</u>
4000</td><td>2900</td><td>1650</td><td>1000</td></tr>
<tr><td>20</td><td>12600</td><td>8500</td><td>6200</td><td><u>4800</u>
4500</td><td>3200</td><td>1800</td><td>1100</td></tr>
<tr><td>25</td><td>13400</td><td>9000</td><td>6800</td><td>5200</td><td>3500</td><td>1950</td><td>1200</td></tr>
<tr><td>30</td><td>14200</td><td>9500</td><td>7400</td><td>5600</td><td>3800</td><td>2100</td><td>1300</td></tr>
<tr><td>35</td><td>15000</td><td>10000</td><td>8000</td><td>6000</td><td>4100</td><td>2250</td><td>1400</td></tr>
<tr><td colspan="8">Notes:
1. Intermediate values should be established by linear interpolation
2. Underlined values are given for sandy soils.
3. If I_D value exceeds 0.67 pile base unit resistance should be increased by 100% if density is established by probing and 60% if other method was used, but pile base unit resistance should be limited to 20000 kN/m^2.
4. For clayey soils with $0.01 < I_P < 0.04$ (loams) and porosity coefficient < 0.8 q_b values of medium dense silty sand should be applied.</td></tr>
</table>

Table 2.10. Pile base unit resistance q_b for bored piles in clayey soils [19]

Pile base depth, m	Pile base unite resistance q_b for clayey soil ($I_P > 0.01$) with I_L, kN/m^2						
	0.0	0.1	0.2	0.3	0.4	0.5	0.6
3	850	750	650	500	400	300	250
5	1000	850	750	650	500	400	350
7	1150	1000	850	750	600	500	450
10	1350	1200	1050	950	800	700	600
12	1550	1400	1250	1100	950	800	700
15	1800	1650	1500	1300	1100	1000	800
18	2100	1900	1700	1500	1300	1150	950
20	2300	2100	1900	1650	1450	1250	1050
30	3300	3000	2600	2300	2000	-	-
40	4500	4000	3500	3000	2500	-	-
Notes: 1. Intermediate values should be established by linear interpolation.							

Pile base unit resistance calculation for bored piles in sandy soils should be performed with given equation [19].

$$q_b = 0.75\alpha_4(\alpha_1\gamma'_z d + \alpha_2\alpha_3\gamma'_v h) \qquad (2.25)$$

where

α_1, α_2, α_3, α_4 – coefficients established according to Table 2.14.

γ'_z – unit weight of soil layer beneath pile base, kN/m^3

γ'_z – average unit weight of soil layers above pile base, kN/m^3

d – pile base diameter, m

h – pile depth, m.

Table 2.11. Coefficients γ_{cs} and γ_{cb} depending on ground conditions and pile installation method [19]

Pile installation method and ground conditions	γ_{cs}	γ_{cb}
Driven piles (installed with impact hammers)	1.0	1.0
Driven piles installed with predrilling if borehole diameter is:		
Equal to pile diameter or cross-section edge length	1.0	0.5
0.05 m shorter than pile diameter or cross-section edge length	1.0	0.6
0.15 m shorter than pile diameter or cross-section edge length	1.0	1.0
Driven piles (installed with vibrators) in given soils:		
Sandy soils	1.0	1.0
Clayey soils with $I_L = 0.5$	0.7	0.9
Clayey soils with $I_L \leq 0$	1.0	1.0
Other piles		
Cast in place displacement piles	1.0	0.7
Bored piles	1.0	0.6
Notes: 1. Coefficients for clayey soils should be established by linear interpolation		

Table 2.12. Coefficient γ_c depending on load direction [19]

Load direction	γ_c
Compression	1.0
Tension (pile shorter than 4 m)	0.6
Tension (pile longer than 4 m)	0.8

Table 2.13. Pile shaft unit resistance $q_{s;i}$ [19]

Soil layer average depth, m	Pile shaft unit resistance $q_{s;i}$, kN/m^2								
	$0.33 < I_D < 0.67$								
	Coarse and medium coarse sand	Fine sand	Silty sand	-	-	-	-	-	-
	Clayey soil ($I_P > 0.01$) with I_L								
	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
1	35	23	15	12	8	4	4	3	2
2	42	30	21	17	12	7	5	4	4
3	48	35	25	20	14	8	7	6	5
4	53	38	27	22	16	9	8	7	5
5	56	40	29	24	17	10	8	7	6
6	58	42	31	25	18	10	8	7	6
8	62	44	33	26	19	10	8	7	6
10	65	46	34	27	19	10	8	7	6
15	72	51	38	28	20	11	8	7	6
20	79	56	41	30	20	12	8	7	6
25	86	61	44	32	20	12	8	7	6
30	93	66	47	34	21	12	9	8	7
35	100	70	50	36	22	13	9	8	7

Notes:
1. Intermediate values should be established by linear interpolation.
2. For clayey soils with $0.01 < I_P < 0.04$ (loams) and porosity coefficient < 0.8 $q_{s;i}$ values of medium dense silty sand should be applied.
3. If I_D value exceeds 0.67 pile shaft unit resistance should be increased by 30%.
4. For clayey soils with $0.01 < I_P < 0.17$ (loams) and porosity coefficient < 0.6 $q_{s;i}$ pile shaft unit resistance should be increased by 15%.

Table 2.14. Coefficients α depending on soil shearing resistance angle [19]

Coefficient	Soil shearing resistance angle φ, °								
	23	25	27	29	31	33	35	37	39
α_1	9.5	12.6	17.3	24.4	34.6	48.6	71.3	108.0	163.0
α_2	18.6	24.8	32.8	45.5	64.0	87.6	127.0	185.0	260.0
α_3 if h/d is:									
4.0	0.78	0.79	0.80	0.82	0.84	0.85	0.85	0.85	0.87
5.0	0.75	0.76	0.77	0.79	0.81	0.82	0.83	0.84	0.85
7.5	0.68	0.70	0.71	0.74	0.76	0.78	0.80	0.82	0.84
10.0	0.62	0.65	0.67	0.70	0.73	0.75	0.77	0.79	0.81
12.5	0.58	0.61	0.63	0.67	0.70	0.73	0.75	0.78	0.80
15.0	0.55	0.58	0.61	0.65	0.68	0.71	0.73	0.76	0.79
17.5	0.51	0.55	0.58	0.62	0.66	0.69	0.72	0.75	0.78
20.0	0.49	0.53	0.57	0.61	0.65	0.68	0.72	0.75	0.78
22.5	0.46	0.51	0.55	0.60	0.64	0.67	0.71	0.74	0.77
> 25	0.44	0.49	0.54	0.59	0.63	0.67	0.70	0.74	0.77
α_4 if d is, m:									
< 0.8	0.34	0.31	0.29	0.27	0.26	0.25	0.24	0.23	0.22
4.0	0.25	0.24	0.23	0.22	0.21	0.20	0.19	0.18	0.17
Notes: 1. Intermediate values should be established by linear interpolation.									

3. Direct pile bearing capacity establishment methods

3.1. Introduction

Unlike indirect pile bearing capacity establishment methods direct methods suggest evaluation and analysis of testing data obtained after pile installation. Like indirect pile bearing capacity establishment methods Eurocode 7 requires design and characteristic resistance values to be determined using specific correlation factors depending on test type and number of test piles.

According to EN 1997-1 two types of tests can be performed on piles subjected to compressive load – static load test and dynamic load test. Static load test is executed according to EN ISO 22477-1, and dynamic load test must comply with EN ISO 22477-4.

Also, as it was mentioned before any pile bearing capacity establishment methods, both direct and indirect, must be proven with static load test, so dynamic load test can be used as proper confirmation to load bearing capacity of a pile only with static load test results on similar piles.

Relatively new direct pile bearing capacity establishment method has been developed and it lies close to dynamic load test but has several differences – rapid load test, which must be executed according to EN ISO 22477-10. Problem with given testing approach is that at the moment measured resistance after execution of this test cannot be evaluated with compliance to EN 1997-1 because actual version of the code does not provide correlation factors and model factors to this specific testing method.

Two major differences between static and dynamic testing approaches can be highlighted – reliability and execution complexity. Static load tests are considered most reliable pile resistance establishment method because it gives real data on pile behaviour under test load in time. This data gives ability to predict pile settlement under specific load and pile creep. Dynamic load testing is data interpretation using empirical, semi-empirical or mechanistical calculation approaches of data established during the test, which means that pile behaviour under the load in time is theoretical value. Execution complexity from the other point of view is disadvantage of static load testing because it requires heavy and large additional equipment. Comparing to that dynamic load tests usually require impact hammer (driven displacement piles most likely can be tested with the same hammer used for pile installation) and measurement devices. Additional advantage of dynamic load testing is duration of the test because dynamic load testing requires only several number of blows (usually 10 blows) to determine necessary data, however static load test usually takes more than 8 hours, considering 8 loading steps and duration of each step 60 minutes.

For offshore piling testing static load testing usually require additional temporary structure constructed around the test pile, which significantly increase cost and execution time of pile testing.

3.2. Static load testing

3.2.1. Execution of static load test

Execution of static load test requires test pile, reaction, force input and measurement devices. It is important to clarify that trial, test and working pile are different terms. Working pile is pile designed for the foundation of the structure. Test pile is the pile, which is subjected to load test to establish resistance, but it can be used as working pile if requirements towards bearing capacity are proven. Trial pile is used for investigation, and it is usually loaded to ultimate load, which is either ground or pile cross-sectional resistance. Test (trial) usually have same geometrical and mechanical properties as working piles. However, for trial piles higher resistance materials can be used to ensure that ultimate load is reached related to ground resistance. Bored trial piles can be constructed with smaller diameter if:

- the ratio between trial and working pile diameters is not less than 0.5;
- trial pile is constructed with same method as working pile;
- trial pile is installed with equipment that the base and shaft resistance can be derived separately [9].

For compressive static load testing as a reaction device several options are possible:

- dead load (kentledge);
- tension piles or anchors;
- an existing structure over the test pile [20].

Loading capability of reaction device should exceed test load for at least 10%. Dead load or kentledge usually consist of steel beam frame on which the weights are loaded. As weights concrete plates of specific space and size are used to ease placing.

Tension piles or anchors are used in situations where placement of dead load reaction system is impossible. Given reaction system usually consists of several piles/anchors working in tension to provide enough tensile resistance needed to compressive load test execution.

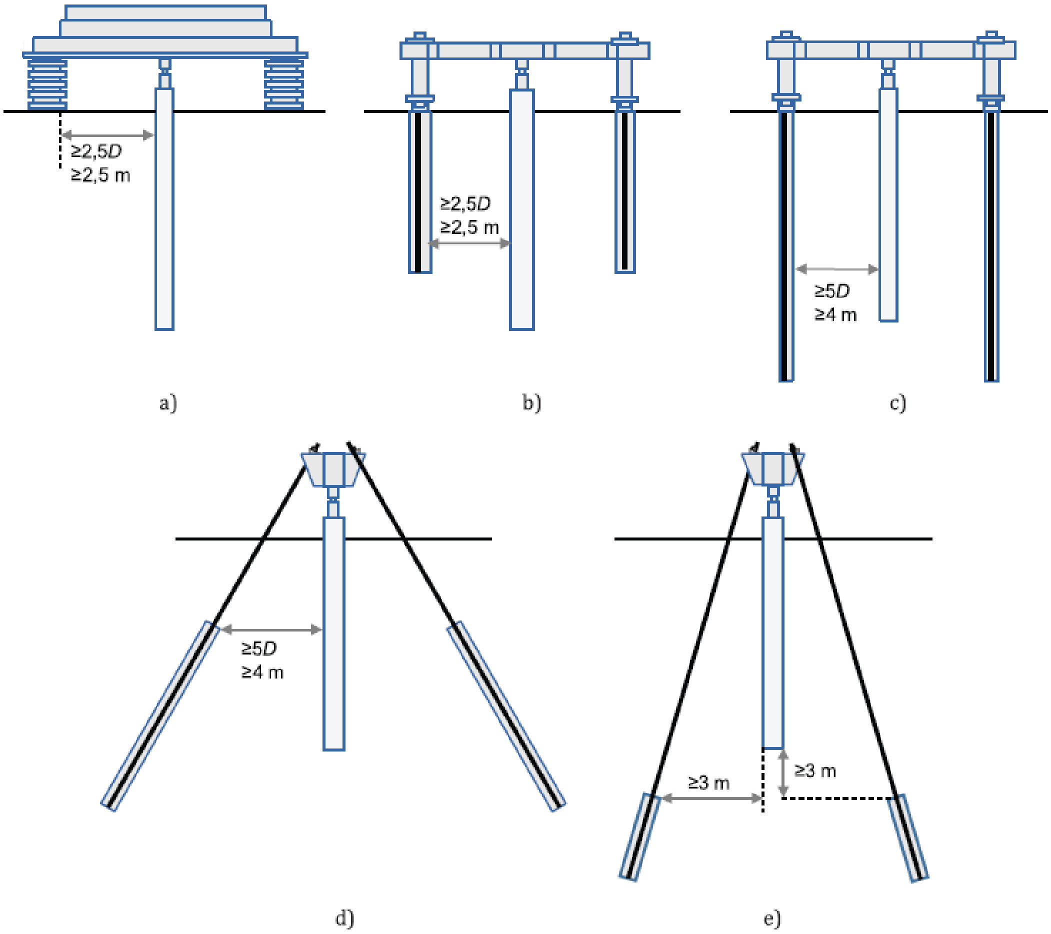

Fig.3.1. Reaction devices a) dead load b, c) tension piles d, e) anchors [20]

Force input device consists of one or several hydraulic jacks that is placed above pile head on distributive steel plate. To control applied load either monometer of hydraulic jack or load cell is used. In both cases devices should be verified and inspected prior the test and both devices should provide precision of load application to 0.5% from test proof load or 10 kN, whichever is greater [20].

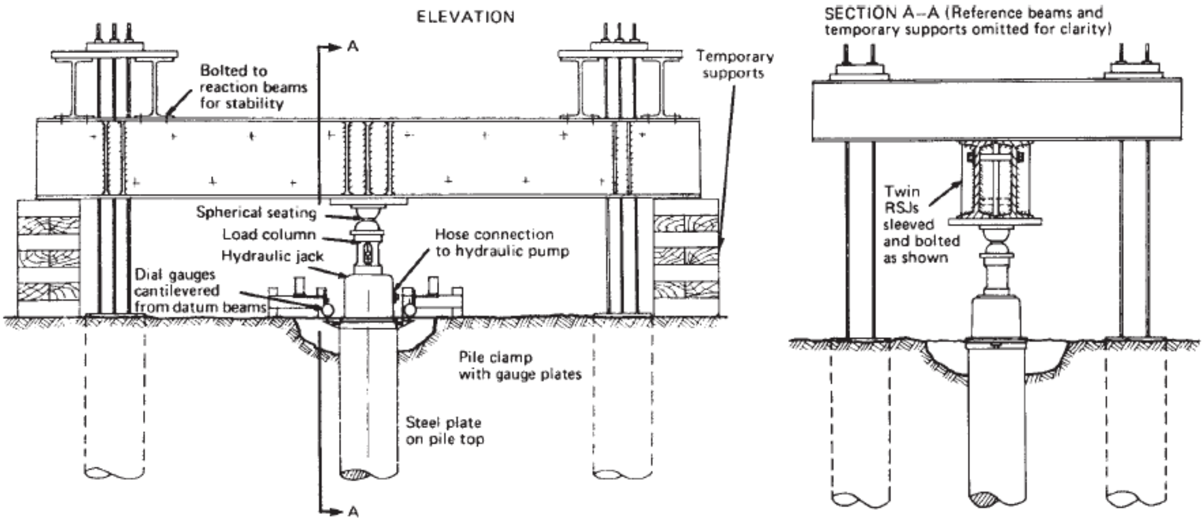

Fig.3.2. Test arraignment using tension piles [4]

Measurement of pile head displacement can be performed with either dial gauges or transducers. At least 3 symmetrically arranged measuring devices should be used. Precision of the readings should meet the requirement at least 0.01 mm. Measuring devices should be places in order to be completely independent from loading device and reaction system. For mentioned purposes reference beam system is used, which shall be placed on supports located on a safe distance away from reaction system. Reference beam should be stiff enough to provide stability of measuring device [20].

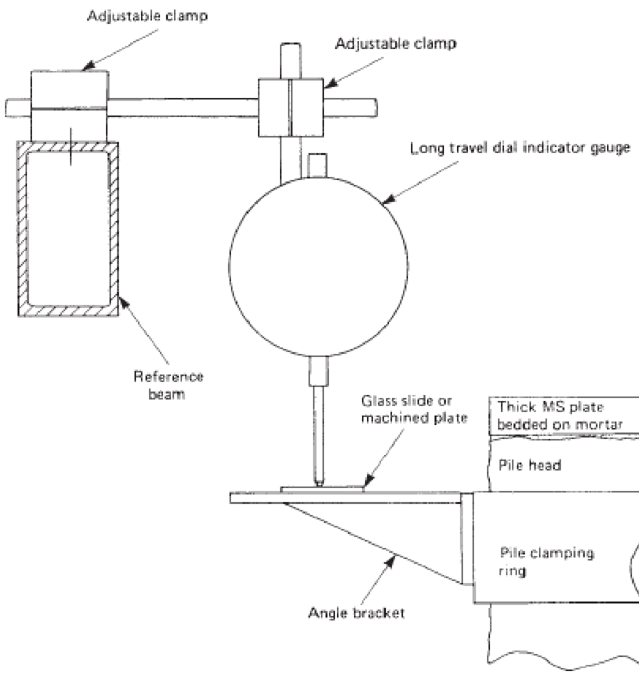

Fig.3.3. Dial gauge measurement device arrangement [4]

Pile construction causes structural changes in soil conditions, which returns to initial with time depending on several parameters, such as, soil type or groundwater level. Therefore, certain time between pile installation and testing must pass. Amount of time needed for soil to restore it initial (working) conditions depends on pile installation method and soil type. Recommended time periods are given in Table 3.1. However, alternative periods can be offered if offered period is based on appropriate justification.

Table 3.1. Recommended time periods between the installation and testing (with static load) of a pile [20]

Soil type	Pile type	Minimum time, days
Coarse soils	All	7
Fine soils	Bored	21
	Displacement	28

Testing of cast in place concrete piles should only begin when the material is strength high enough to sustain testing loads.

Loading procedure consists of at least 8 loading and 4 unloading steps. Step duration is different for loading and unloading procedure. Each loading step takes at least 60 minutes (duration can be reduced on first loading steps if creep requirement is reached), but unloading steps are usually not longer than 10-15 minutes. Exception is last unloading step when load is already 0 kN and pile head displacement measurements should be made for 30 minutes. Loading can be performed either in one or multiple cycle procedure. One cycle procedure requires pile loading prior to proof load then unloading it. Multiple cycle procedure requires pile loading in given order – loading to characteristic load in at least 4 steps, unloading in at least 2 steps, loading to proof load in at least 8 steps and unloading in at least 4 steps.

3.2.2. Pile load bearing capacity establishment from static load test results

According to EN 1997-1 characteristic and design values of pile resistance can be established from measured values by applying correlation and partial safety factors. Given values are calculated by Equation (3.1) and Equation (3.2) [9].

$$R_{c;d} = \frac{R_{c;k}}{\gamma_t} \qquad (3.1)$$

where

$R_{c;d}$ – pile design bearing capacity (compressive resistance), kN

$R_{c;k}$ – pile characteristic bearing capacity (compressive resistance), kN

γ_t – total partial factor for compressive resistance.

$$R_{c;k} = \left\{ \frac{(R_{c;m})_{mean}}{\xi_1} ; \frac{(R_{c;m})_{min}}{\xi_2} \right\} \qquad (3.2)$$

where

$(R_{c;m})_{mean}$ – mean measured total pile bearing capacity, kN

$(R_{c;m})_{min}$ – minimal measured total pile bearing capacity, kN

ξ_1 and ξ_2 – correlation factors which depend on number of static load tests performed given in Table 3.2.

Table 3.2. Correlation factors ξ for characteristic resistance establishment from static load test results [9]

Number of tests	1	2	3	4	≥ 5
ξ_1	1.40	1.30	1.20	1.10	1.00
ξ_2	1.40	1.20	1.05	1.00	1.00

Measured total pile bearing capacity is established on analysis of static load test results. Suitable proof load P_p to confirm desired pile resistance can be calculated by Equation (3.3).

$$P_p = (R_{c;m})_{mean} = R_{c;d}\xi_1\gamma_t \qquad (3.3)$$

where

P_p – proof load needed to confirm desired pile bearing capacity, kN.

Evaluation of static load test results is performed on load- displacement curve analysis. Two main simple criteria which can be applied to most cases are:

- the load at which displacement continues to increase without further increase in load (Point A Fig.3.4.);
- the load causing displacement of 10% of the pile base diameter or equivalent diameter for non-circular piles (Point B Fig.3.4.) [1].

Except to main criteria other failure criteria can be recognized:

- the load beyond which increase in displacement occurs disproportionate to the increase in load (Point C and Point D Fig.3.4);
- the load that produces causes plastic deformations or plastic yielding (Point E Fig.3.4);
- the load at the cross point of tangent lines drawn through flatter and steeper parts of load- displacement curve (Point F Fig.3.4.) [1].

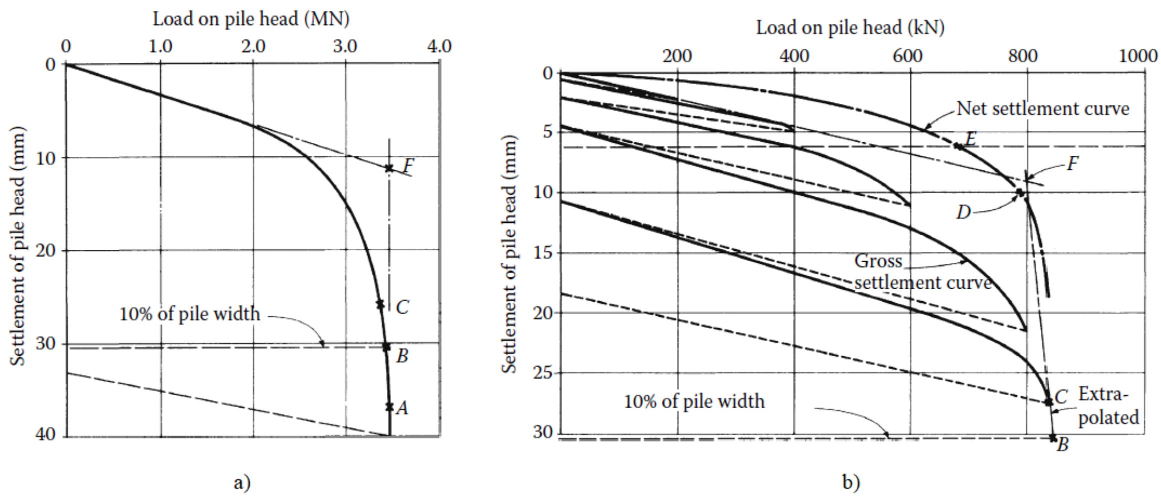

Fig.3.4. Typical load-displacement curves for a) one cycle static load test b) multiple cycle static load test [1]

Additionally, to parameters established directly from load- displacement curve pile creep ratio can define bearing capacity. Creep ratio is using Equation (3.4) and its definition is shown in Fig.3.5. Limit value of creep ratio is 2 mm, but it can be specified by expert in geotechnics [21].

$$k_s = \frac{s_b - s_a}{\lg (t_b/t_a)} \qquad (3.4)$$

where

k_s – creep ratio, mm

s_b – displacement at the end of observation period, mm

s_a – displacement at the beginning of observation period, mm

t_b – time at the end of observation period, min

t_a – time at the beginning of observation period, min.

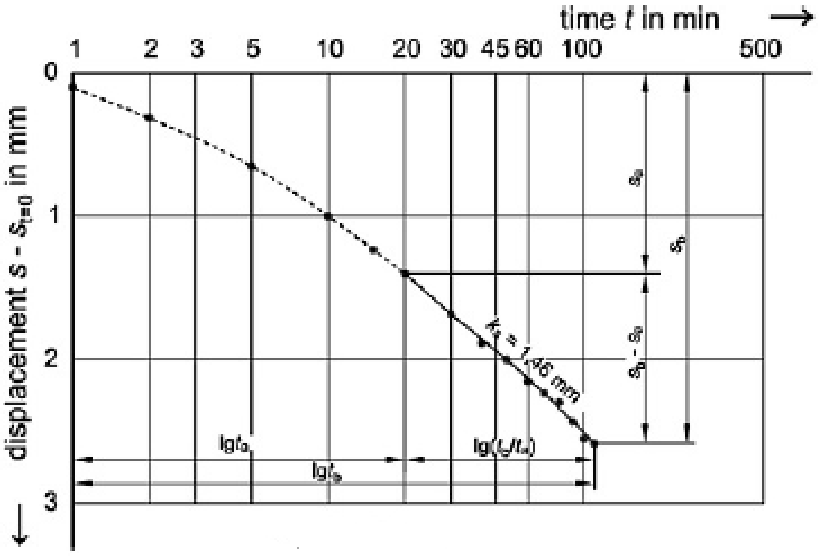

Fig.3.5. Establishment of creep ratio [21]

3.3. Dynamic load testing

3.3.1. Execution of dynamic load test

Dynamic pile testing has become normal practice in piling. It is possible to monitor impact hammer and driving stresses during the installation of pile. Due to fact that soil resistance is variable in time evaluation of long-term soil resistance is usually evaluated during restrike several days later. Dynamic testing is successfully applied not only on driven displacement piles but also on bored and cast in situ displacement piles [22].

Execution of dynamic load test depends on selected resistance establishment approach, which are:

- dynamic impact testing;
- pile driving formulas;
- wave equation analysis.

Difference can be established between dynamic impact testing and pile driving formulas with wave equation analysis because of difference in execution and required measurements. Dynamic impact testing requires specific equipment for strain, acceleration, and displacement measurements with high sampling rate because of short time of loading. Wave equation analysis and pile driving formulas requires only measurement of set per blow, which is usually taken as average value for specific number of blows. Set per blow means displacement of pile per one hit by impact hammer.

Nevertheless, wave equation analysis is mentioned as one of possible resistance establishment approaches, given thesis will specify on dynamic impact testing and pile driving formula. This decision is justified because dynamic impact testing is based on wave equation analysis, and it will not be included in practical part of the thesis.

All approaches require loading equipment that can generate adequate force and impact energy that fulfils the requirements. For execution of dynamic load tests impact hammers (hydraulic and diesel) are usually used. The mass of impact hammers drop mass (expressed in force units) must exceed 2% of the design resistance [23].

Fig.3.6. Junttan hydraulic impact hammer [24]

As for static load tests sufficient time has to pass for soil to regain its normal working conditions. According to EN ISO 22447-4 driving process of prefabricated piles can be considered as dynamic testing if all required measurements are made, but it is vitally important to evaluate ground conditions because different soils cause different effects of pile bearing capacity, for example friction pile resistance in clayey soils usually decreases with time. For this reason, re-driving or execution of dynamic load test after specific time after installation of the pile must be considered. Rest period for bored or cast in situ piles is given in Table 3.3.

Table 3.3. Recommended time periods between the installation and testing (with dynamic load) of bored or cast in situ pile [23]

Test pile type	Soil type	Minimum time, days
Trial	Non-cohesive	7
	Cohesive	21
Working	Non-cohesive	5
	Cohesive	14

Different source (ГОСТ 5686) states that soil resurrection period depends on different soil conditions, such as, soil composition, properties, and condition. Recommended time periods for all pile types are given in Table 3.4.

Table 3.4. Recommended time periods between the installation and testing (with dynamic load) of piles according to ГОСТ 5686

Soil type	Minimum time, days
Sandy soils	3
Clayey and mixed soils	6
Coarse dense sandy (gravelly) and stiff clayey soils	1
Water-saturated, fine and silty sands	10
Clayey soils with high plasticity	20

For bored and cast in situ piles it is important to consider that concrete strength bus be sufficient to sustain stresses caused by impact hammer blows.

It should be mentioned that impact energy can be increased either by using heavier impact hammer or increasing drop height, however as it is seen in Fig.3.7. hammer to pile weight ratio influences distribution of energy. As it is seen creating the same amount of impact energy by lighter hammer causes more energy losses (inertia loss and temporary compression loss). So, it is advised to increase impact energy by increasing hammer weight to improve drivability, decrease stresses in pile itself and cause less damage to pile head during the driving.

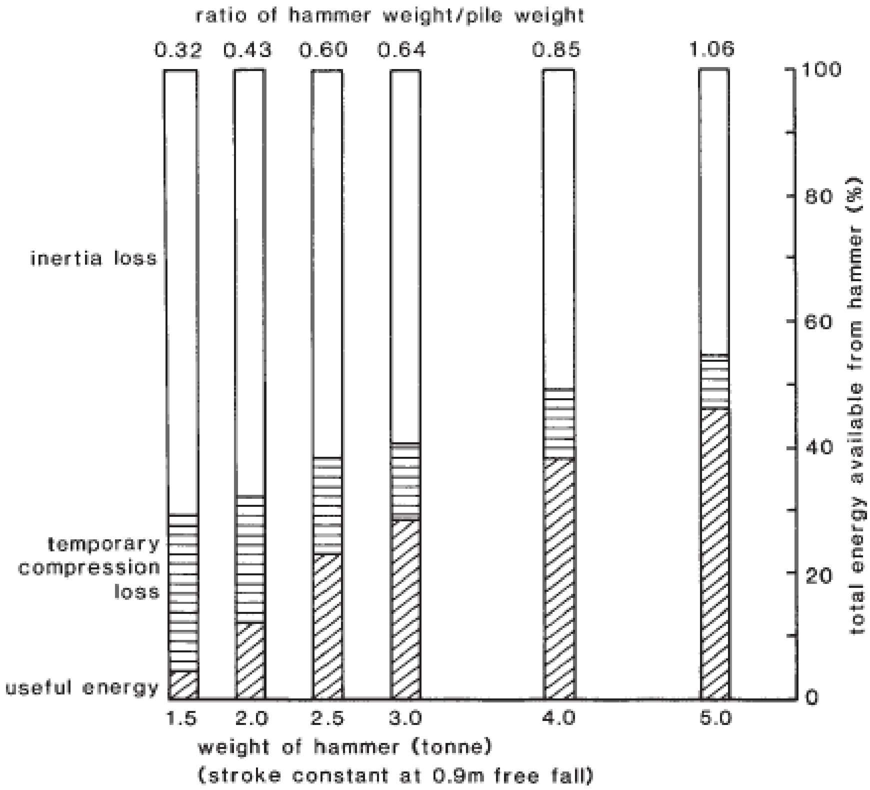

Fig.3.7. Distribution of energy during pile driving depending on hammer to pile weight ratio [4]

3.3.2. Dynamic impact testing

During execution of dynamic impact test minimum of 3 parameters must be directed directly:

- the strain at the pile head;
- the acceleration of the pile head;
- the permanent displacement of pile head per dynamic impact (set per blow) or (if multi-blow dynamic testing is performed) total head displacement shall be measured [23].

According to EN 22477-4 results of dynamic impact testing can be evaluated by 3 methods:

- Direct closed form solution based on soil dependent damping values (CASE and TNO methods);
- Signal matching;
- Multi-blow dynamic load testing approach [23].

Direct closed form solutions unlike signal matching and multi-blow dynamic load testing approach can be analyzed in relatively simple way without complicated mathematical models or regression analysis, therefore only those methods are described below.

Total soil resistance is given in Equation (3.5). In given equation resulting total resistance (R_{tot}) is established with same expression for both methods (CASE and TNO), however dynamic resistance is established differently. CASE method assumes that all soil resistance (both tip and friction resistance) is located at pile tip. TNO method assumes that friction works at one point along the shaft and toe resistance is located at pile toe. Multi-blow dynamic testing method considers that dynamic soil resistance is a function of the permanent displacement [23].

$$R_{tot} = \frac{1}{2}(F_1 + Z\nu_1) + \frac{1}{2}(F_2 - Z\nu_2) \qquad (3.5)$$

where

R_{tot} – total soil resistance, kN

F_1 – force calculated from strain measurement by Equation (3.6) at time t_1, kN

F_2 – force calculated from strain measurement by Equation (3.6) at time t_2, kN

Z – proportionality factor calculated by Equation (3.7), kN/m/s

ν_1 – velocity at pile head calculated from acceleration measurement by Equation (3.8) at time t_1, m/s

ν_2 – velocity at pile head calculated from acceleration measurement by Equation (3.8) at time t_2, m/s

t_1 – time at first force peak of force-time diagram, s

t_2 – time needed to impact wave to travel from pile head to toe and back calculated by Equation (3.9), s

$$F_{(t)} = \varepsilon(t)EA \qquad (3.6)$$

where

F(t) – force at time t, kN

ε(t) – measured strain at time t

E – pile modulus elasticity, kN/m^2

A – pile cross-section area, m^2

$$Z = \frac{AE}{c} \qquad (3.7)$$

where

c – wave propagation velocity in the pile material calculated by Equation (3.10), m/s

$$\nu_{(t)} = \int a(t)\,dt \qquad (3.8)$$

where

ν(t) – velocity at time t, m/s

a(t) – measured acceleration at time t, m/s^2

$$t_2 = t_1 + \frac{2L}{c} \qquad (3.9)$$

where

L – length of the pile, m

$$c = \sqrt{\frac{E}{\rho}} \qquad (3.10)$$

where

ρ – density of the pile material, kg/m^3

Static soil resistance is calculated by Equation (3.11)

$$R_{stat} = R_{tot} - R_{dyn} \qquad (3.11)$$

where

R_{stat} – static soil resistance, kN

R_{dyn} – dynamic soil resistance, kN

Dynamic resistance according to CASE method is calculated by Equation (3.12) [23].

$$R_{dyn} = J_c Z \nu_b \qquad (3.12)$$

where

J_c – damping coefficient depending on soil type according to Table 3.5

ν_b – penetration velocity of pile base calculated by Equation (3.13), m/s

Table 3.5. Damping coefficients for CASE method [23]

Soil type	J_c
Sand	0.05-0.20
Silty sand	0.15-0.30
Silt	0.20-0.45
Silty clay	0.40-0.70
Clay	0.6-1.10

$$\nu_b = \nu_1 + \frac{(F_1 - R_{tot})}{Z} \qquad (3.13)$$

Dynamic resistance according to TNO method is divided between pile base and pile shaft and it is calculated by Equation (3.14) [23].

$$R_{dyn} = R_{b,dyn} + R_{s,dyn} \qquad (3.14)$$

where

$R_{b,dyn}$ – base dynamic resistance calculated by Equation (3.15), kN

$R_{s,dyn}$ – shaft dynamic resistance calculated by Equation (3.16), kN

$$R_{b,dyn} = \nu_b A_b C_b \qquad (3.15)$$

where

ν_b – same as in Equation (3.13), m/s

A_b – pile base cross-section area, m^2

C_b – damping coefficient for pile base resistance according to Table 3.6, $kN/m^2/m/s$

$$R_{s,dyn} = \nu_s A_s C_s \qquad (3.15)$$

where

ν_s – velocity at the pile shaft calculated by Equation (3.16), m/s

A_s – shaft area covered in soil, m^2

C_s – damping coefficient for pile shaft resistance according to Table 3.6, $kN/m^2/m/s$

Table 3.6. Damping coefficients for TNO method [23]

Soil type	C_s, $kN/m^2/m/s$	C_b, $MN/m^2/m/s$
Sand	2-10	0.4-2.0
Sandy silt	5-15	1.0-3.0
Silt	10-25	2.0-5.0
Silty clay	20-40	4.0-8.0
Clay	25-50	5.0-10.0

$$\nu_s = \frac{1}{2}\left(\nu_1 + \frac{F_1}{Z}\right) - \frac{1}{2}\left(\frac{F_3}{Z} - \nu_3\right) \qquad (3.16)$$

where

F_3 – force calculated by Equation (3.6) at time t_3, kN

ν_3 – velocity calculated by Equation (3.8) at time t_3, m/s

t_3 – time representing the maximum difference between force and velocity (less than t_2 within the time range 2L/c)

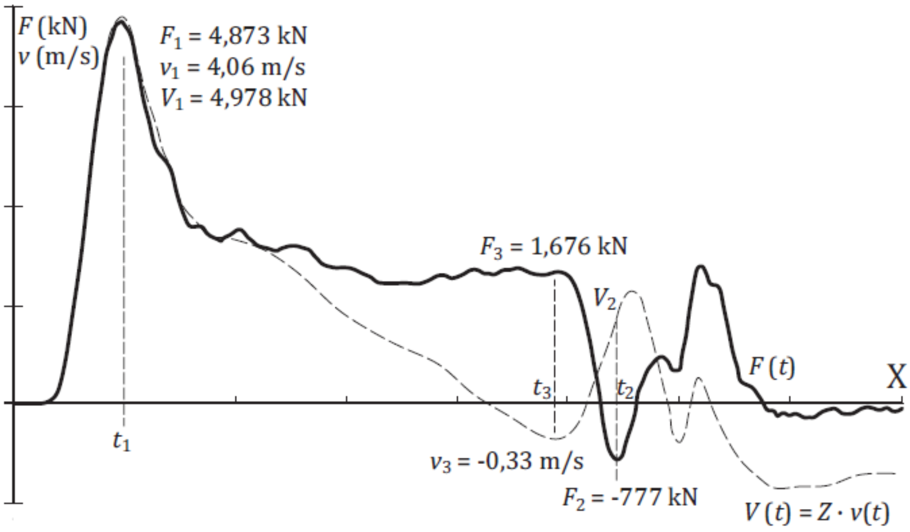

Fig.3.8. Explanation of points t_1, t_2 and t_3 [23]

3.3.3. Pile driving formulas

Pile driving formulas have been widely used historically due to it simplicity and easy application. For most of the formulas only several easily determinable parameters are needed, such as, impact energy and set per blow. Most of the formulas are semi-empirical containing parameters that have been determined experimentally.

Considering gross assumptions that were made to simplify analysis, correlation between static load testing and pile driving formulas is very poor. Pile driving formulas ignore soil type, hammer efficiency and actual transmitted energy (from potential to kinetic) [25].

However, many formulas contain parameters that consider soil parameters, hammer efficiency and other factors that can cause inaccuracies. Approach provided in EN ISO 22477-4 suggests establishing correlation with static load testing to improve accuracy, though approach used in Russia (Gersivanov formula) is based on experimentally determined parameters for different types of hammers and soils. It is common practice that pile bearing capacity established by pile driving formulas or any other dynamic testing approach is reduced by higher safety coefficients.

One of the most used pile driving formulas through history is Hiley formula given in Equation (3.17). This formula has several variations and empirical coefficients vary in different sources. In this thesis Dawson Engineering interpretation of Hiley formula is used [26].

$$R = \frac{4E}{(s + 2.54)} \qquad (3.17)$$

where

R – pile bearing capacity, kN

E – hammer impact energy, kg·m

s – set per blow, mm

Numerical factors given in Equation (3.17) already contain safety factor of 2 and considers impact hammer efficiency of 80%. Given formula is an example of major simplifications where formula suggests that pile bearing capacity depends only on impact energy and set per blow.

Other driving formula given as Equation (3.18) suggested in 1957 by Sorensen and Hansen states that pile resistance depends on 5 factors – the pile driver efficiency, hammer weight, hammer weight drop height, set per blow, pile length, pile cross-section area and pile modulus of elasticity [27].

$$R = \frac{\eta W h}{s + \frac{1}{2}\sqrt{\frac{2\eta W h L}{AE}}} \qquad (3.18)$$

where

R – pile bearing capacity, kN

η – pile driver efficiency

W – hammer impact weight, t

h – hammer weight drop height, m

s – set per blow, m

A – pile cross-section area, m^2

E – pile modulus of elasticity, kN/m^2

L – pile length, m

Given formula also theoretically considers elastic deformations during impact of a hammer as an addition to set per blow, which reduces resistance. However, given approach does not consider soil type and its possible effects.

Approach provided in Russian standard СП 24.13330 is based on Gersivanov formula. Gersivanov formula is similar to Danish formula, however it has differences depending on set per blow. It is considered that if set per blow is more than 2 mm then elastic deformation of pile caused by hammer impact can be neglected. Although, if set per blow is less than 2 mm suggestion is to either choose impact hammer with higher impact energy or use devices that can measure elastic deformations of the pile.

Gersivanov formula is given in Equation (3.18) and Equation (3.19) [28].

$$R = \frac{\eta A}{2}\left(\sqrt{1 + \frac{4E_d}{\eta A s_p} * \frac{m_1 + \varepsilon^2(m_2 + m_3)}{m_1 + m_2 + m_3}} - 1\right) \qquad \text{if } s_p \geq 2\text{ mm} \qquad (3.18)$$

$$R = \frac{1}{2\theta} * \frac{2s_p + s_{el}}{s_p + s_{el}}\left(\sqrt{1 + \frac{8E_d(s_p + s_{el})}{(2s_p + s_{el})^2} * \frac{m_4}{m_4 + m_2}\theta} - 1\right) \qquad \text{if } s_p < 2\text{ mm} \qquad (3.19)$$

where

R – pile bearing capacity, kN

η – empirical coefficient selected according to Table 3.7. considering pile material, kN/m^2

A – pile cross-section area, m^2

E_d – hammer design impact energy according to Table 3.8. considering impact hammer type, kJ (kNm)

s_p – set per blow (permanent pile displacement per blow), m

s_{el} – elastic deformation of pile caused by impact hammer blow, m

ε – impact recovery coefficient (ε = 0.2)

m_1 – total weight of impact hammer, kg

m_2 – total pile weight, kg

m_3 – weight of additional equipment used (driving caps, driving extenders etc.), kg

m_4 – weight of hammer impact part, kg

θ – coefficient calculated by Equation (3.20), 1/kN

$$\theta = \frac{1}{4}\left(\frac{n_b}{A} + \frac{n_f}{A_f}\right)\frac{m_4}{m_4 + m_2}\sqrt{2g(H - h)} \qquad (3.20)$$

where

n_b – correlation coefficient between dynamic and static impact for pile base resistance (n_b = 0.00025), s·m/kN

n_f – correlation coefficient between dynamic and static impact for pile shaft resistance (n_f = 0.025), s·m/kN

g – acceleration due to gravity (g = 9.81), m/s^2

H – hammer impact part drop height, m

h – first rebound height of diesel hammer after the blow (h = 0 for different type hammers), m

A_f – pile friction area, m^2

Table 3.7. Coefficient η values [28]

Pile type	η, kN/m^2
Concrete pile	1500
Timber pile	800-1000

Table 3.8. Calculation of design impact energy [28]

Impact hammer type	E_d, kNm (kJ)
Hydraulic or single impact hammer	E_d = GH
Diesel hammer	E_d = G(H-h)
G – weight of impact hammer moving part, kN H – drop height of impact hammer, m h – first rebound height of diesel hammer after the blow, m	

European standard EN ISO 22477-4 suggests approach that establishes correlation directly using results of static load test results. By given approach first pile of a pile group is tested with both static and dynamic load tests. Calculation of correlation coefficient based on static load test and dynamic load test results that contain set per blow and impact energy can be performed by Equation (3.21) [23].

$$\eta = R_{stat} \frac{s_p + s_{el}}{C_r E_k} \qquad (3.21)$$

where

η – correlation coefficient between results of static load testing and dynamic load testing

R_{stat} – pile bearing capacity approved by static load testing, kN

s_p – set per blow (permanent pile displacement per blow), m

s_{el} – elastic deformation of pile caused by impact hammer blow, m

C_r – efficiency coefficient calculated by Equation (3.22) or taken from impact hammer datasheet

E_p – potential energy used for dynamic testing calculated by Equation (3.23)

It should be mentioned that efficiency of impact hammer can be affected by various factors, so for precision improvement transmitted energy should be calculated directly by results of acceleration measurements at pile head during the impact.

$$C_r = \frac{E_{tr}}{E_p} \qquad (3.22)$$

where

E_{tr} – transmitted energy to pile head during the impact, kJ (kNm)

$$E_p = mgh \qquad (3.23)$$

where

m – mass of impact hammer moving part, t

g – acceleration due to gravity (g = 9.81), m/s^2

h – drop height of impact hammer moving part, m

Pile bearing capacity is established by Equation (3.24).

$$R = \frac{\eta E_p C_r}{s_p + s_{el}} \qquad (3.24)$$

where

R – pile bearing capacity, kN

3.3.4. Pile load bearing capacity establishment from dynamic load test results

According to EN 1997-1 characteristic and design values of pile resistance can be established from measured values by applying correlation and partial safety factors. Given values are calculated by Equation (3.25) and Equation (3.26) [9].

$$R_{c;d} = \frac{R_{c;k}}{\gamma_t} \qquad (3.25)$$

where

$R_{c;d}$ – pile design bearing capacity (compressive resistance), kN

$R_{c;k}$ – pile characteristic bearing capacity (compressive resistance), kN

γ_t – total partial factor for compressive resistance.

$$R_{c;k} = \left\{\frac{(R_{c;m})_{mean}}{\gamma_M \xi_1}; \frac{(R_{c;m})_{min}}{\gamma_M \xi_2}\right\} \qquad (3.26)$$

where

$(R_{c;m})_{mean}$ – mean measured total pile bearing capacity, kN

$(R_{c;m})_{min}$ – minimal measured total pile bearing capacity, kN

ξ_5 and ξ_6 – correlation factors which depend on number of dynamic load tests performed given in Table 3.9.

γ_M – model factor depending on type of dynamic load test given in Table 3.10

Table 3.9. Correlation factors ξ for characteristic resistance establishment from dynamic load test results [9]

Number of tests	≥ 2	≥ 5	≥ 10	≥ 15	≥ 20
ξ_5	1.60	1.50	1.45	1.42	1.40
ξ_6	1.50	1.35	1.30	1.25	1.25

Table 3.10. Model factors for dynamic load testing [9]

Type of dynamic load test	γ_M
Dynamic load test with signal matching	0.85
Pile driving formula measuring s_{el}	1.10
Pile driving formula without measuring s_{el}	1.20
Other dynamic load test	1.00

3.4. Rapid load testing

3.4.1. Execution of rapid load test

Rapid load testing is similar to dynamic load testing by its execution principles. In both testing approaches dynamic impact is used as main variable in determination of pile bearing capacity.

The main difference between two testing approaches is in duration of the dynamic loading on the pile head, relative to the duration that a stress wave needs to travel from the pile head to the pile base. Dynamic load is considered rapid load if these criteria are met:

- the duration of loading should be significantly longer than a stress wave needs to travel from pile head to base. The load duration should be that long that a tensile stress wave in a pile is not induced;
- the loading is continuously increasing the decreasing in a smooth manner [29].

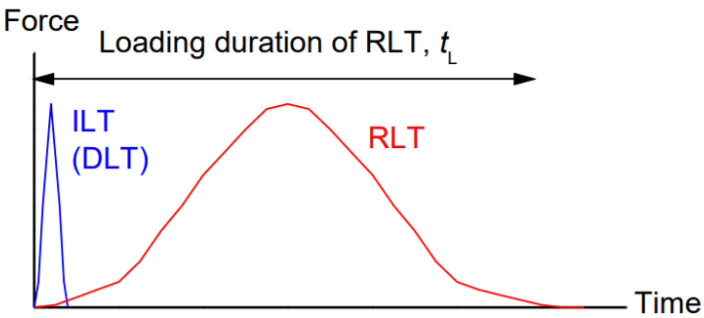

Fig.3.9. Loading duration comparison between rapid load testing (RLT) and dynamic load testing (DLT) [30]

Criteria for dynamic load to be considered as rapid load is given in Equation (3.27) [31].

$$10 < \frac{t_f c_p}{L} \leq 1000 \qquad (3.27)$$

where

t_f – duration of the load application, s

c_p – velocity of the stress wave in the pile calculated by Equation (3.28), m/s

L – length of the pile, m

$$c_p = \sqrt{\frac{E}{\rho}} \qquad (3.28)$$

where

E – modulus of elasticity of the pile, N/m^2

ρ – pile density, kg/m^3

During the execution of rapid load test a minimum of 3 variables should be measured directly relative to time:

- the force applied to the pile head;
- the displacement of the pile head;
- the acceleration of the pile head [31].

Because of the direct measurements of applied force and acceleration cannot be measured without special equipment, such as, load cells and accelerometers attached to the pile head, execution of rapid load test cannot be simplified as dynamic load test when only measurement of set per blow can be measured directly.

As prior static and dynamic load testing sufficient time period between installation of the pile and test execution date has to pass to soil regain its normal conditions. Time periods are given in Table 3.11.

Table 3.11. Recommended time periods between the installation and testing (with rapid load) of piles [31]

<table>
<tr><th>Test pile type</th><th>Soil type</th><th>Pile type</th><th>Minimum time, days</th></tr>
<tr><td rowspan="3">Trial</td><td>Non-cohesive</td><td>All</td><td>7</td></tr>
<tr><td rowspan="2">Cohesive</td><td>Bored</td><td>21</td></tr>
<tr><td>Driven</td><td>35</td></tr>
<tr><td rowspan="3">Working</td><td>Non-cohesive</td><td>All</td><td>5</td></tr>
<tr><td rowspan="2">Cohesive</td><td>Bored</td><td>14</td></tr>
<tr><td>Driven</td><td>21</td></tr>
</table>

However, for execution of rapid load test specific impact hammers are needed. Normally impact hammers used for installation of the pile use either potential energy (hydraulic hammers) or diesel engine principle (diesel hammers), and both types of these hammers create short loading, which does not meet criteria of loading duration. For that reason, special impact hammers with extended loading length should be used.

3.4.2. Pile load bearing capacity establishment from rapid load test results

To determine pile bearing capacity from the results of rapid load test data of directly measured parameters should be evaluated. For analysis needed data is seen in Fig.3.10. In Fig.3.10. force (a), displacement (b), velocity (c) calculated from acceleration (d) values depending on time are seen.

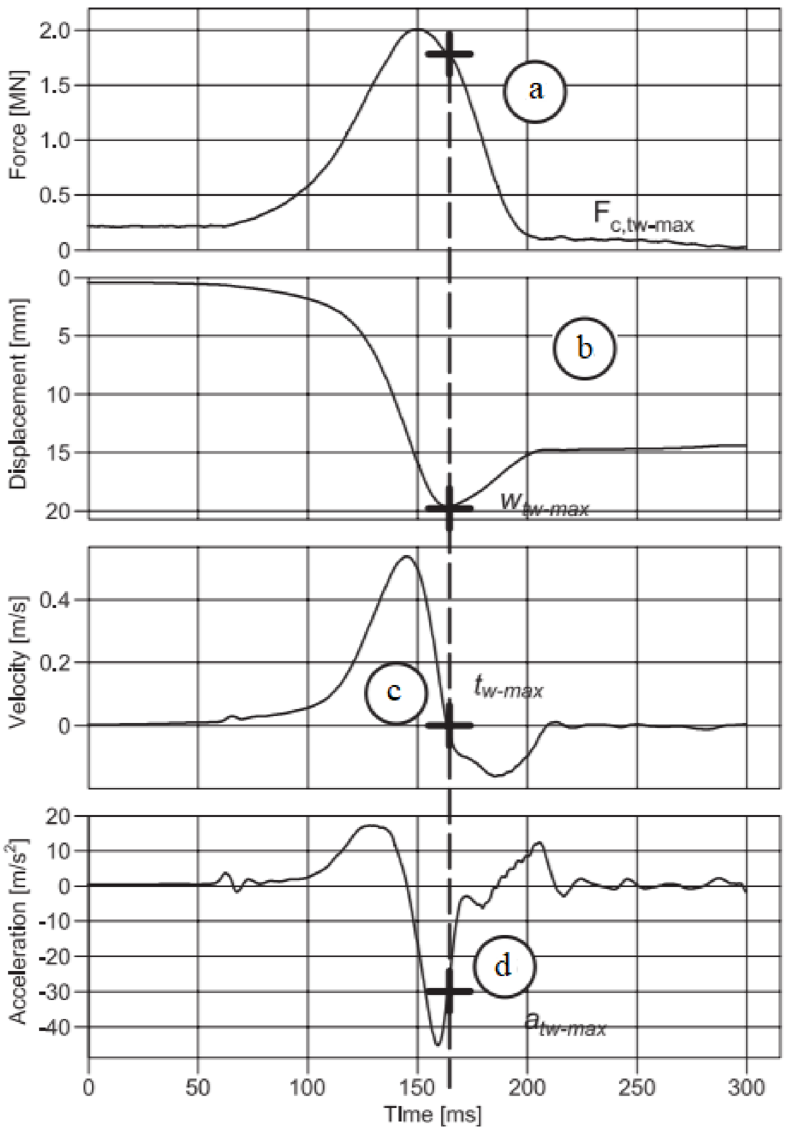

Fig.3.10. Data established during rapid load testing [31]

From Fig.3.10. time value (t_{w-max}) when velocity is zero or close to it should be established to determine an unloading point. Afterwards value of recorded force ($F_{c,tw-max}$) and acceleration (a_{tw-max}) at time t_{w-max} should be determined. With given values the inertia corrected bearing capacity ($R_{c,\ ic,\ tw-max}$) can be calculated using Equation (3.29) [31].

$$R_{c,ic,tw-max} = F_{c,tw-max} - (\rho A L a_{tw-max}) \qquad (3.29)$$

where

$R_{c,\ ic,\ tw-max}$ – inertia corrected pile bearing capacity, kN

$F_{c, tw-max}$ – recorded force at time t_{w-max}, kN

ρ – pile material density, kg/m^3

A – pile cross-section area, m^2

L – pile length, m

a_{tw-max} – acceleration at time t_{w-max}, m/s^2

Inertia corrected pile bearing capacity shall be corrected with empirical factor dependent on soil type using Equation (3.30) [31].

$$R_{c,m} = R_{c,corrected} = \eta R_{c,ic,tw-max} \qquad (3.30)$$

where

$R_{c,m}$ – measured pile bearing capacity, kN

$R_{c, corrected}$ – considering soil type corrected pile bearing capacity, kN

η – empirical factor dependent on soil type (0.66 for clay and 0.94 for sand)

From the results of rapid load test load-displacement curve can be established. The procedure of inertia corrected load-displacement curve consists of 2 steps:

- determination of initial stiffness parameter (p);
- determination of remaining the hyperbola formula parameter (q) [32].

Firstly, it is necessary to draw inertia corrected load-displacement curve as a function of displacement w according to Equation (3.31) and Fig.3.11. (a) [31].

$$R_{c,ic,tw-max}(w) = F_{c,tw-max}(w) - [\rho A L a_{tw-max}(w)] \qquad (3.31)$$

where

w – displacement as variable, mm

Afterwards, inertia corrected load-displacement curve is transferred into hyperbola coordinate system as it seen in Fig.3.11. (b). To determine parameter p best fit line should be drawn Fig.3.11. (c) and parameter p can be determined directly from the graph Fig.3.11. (d) [31].

Lastly, factor q can be established for corrected pile bearing capacity ($R_{c,\ corrected}$) and inertia corrected pile bearing capacity ($R_{c,\ ic,\ tw\text{-}max}$) using Equation (3.32) [31].

$$q = \frac{1}{R - \frac{p}{w_{tw-max}}} \qquad (3.32)$$

where

q – remaining hyperbola formula parameter, 1/kN

R – pile bearing capacity ($R_{c,\ corrected}$ or $R_{c,\ ic,\ tw\text{-}max}$ depending on which graph is determined), kN

p – initial stiffness parameter, mm/kN

$w_{tw\text{-}max}$ – pile head displacement at time $t_{w\text{-}max}$, mm

When factors p and q are determined corrected load-displacement curve can be drawn using Equation (3.33) as it is seen in Fig.3.11. (e) and (f) [31].

$$R(w) = \frac{w}{p + (qw)} \qquad (3.33)$$

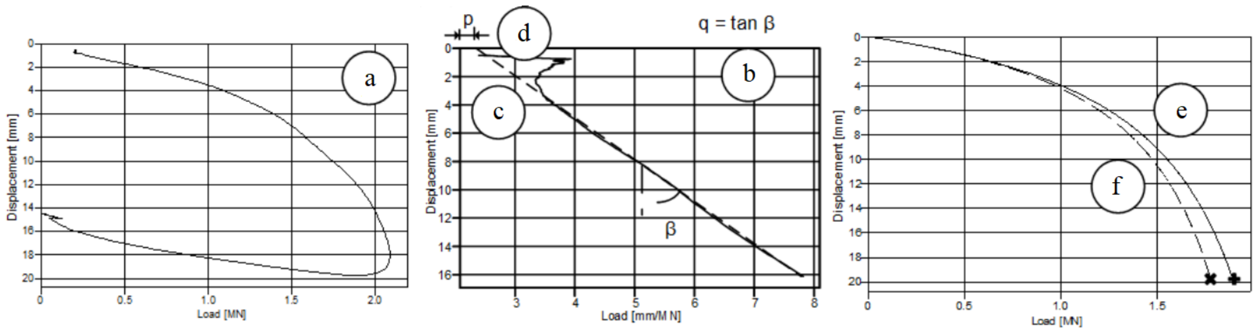

Fig.3.11. Determination of load-displacement curve [31]

4. Combining different testing methods

4.1. Previous experience

Combining static and dynamic load pile bearing capacity testing approaches has been done through years at various construction sites all over the world. Combination of given testing methods gives advantages from both – high reliability from static load test and execution speed and simplicity from dynamic load test.

Comparison of static and dynamic load tests performed on different length and type piles in India by C.H. Solanki and his research group. Research consisted of pile testing with both methods and main conclusions are:

- load-displacement curve established by results of dynamic testing show good correlation with static load test results when the load is low. When the load increases, the dynamic load test may underestimate the displacement;
- dynamic load test should be calibrated by at least one static load test;
- combination of dynamic load test with static load test can improve cost-efficiency of pile testing program [33].

Similar comparison was done during construction at project site in Ain Sukhna, Egypt where Ø1219 mm open-end steel piles were installed. Test program consisted of dynamic and static load testing of piles to establish correlation and optimize pile length. Research concluded high correlation between static and dynamic load test results for specific site and test results allowed to significantly reduce pile length therefore increase cost-efficiency [34].

During the construction of offshore wind farm in the German Baltic Sea (water depth around 40 m), where open-end steel piles Ø1370x40 mm were driven into the ground up to 31 m, pile testing program consisted of combination of static and dynamic load testing. As it was mentioned previously, execution of offshore static load testing has additional difficulties, such as, application of reaction system required to reach ultimate pile bearing capacity, so combination of testing approaches was performed in order to extrapolate results of static load testing. In given project dynamic load testing provided additional information to improve confidence in extrapolation of static load test results. Additionally, dynamic load test was executed during installation of piles and 10 weeks after (together with static load tests) to establish difference in soil resistance depending on time. Conclusion was made that soil resistance in this specific site significantly increases in time. Result of combined testing that confirmed pile bearing capacity turned out in reduction of pile lengths, saving around 8000 t of steel [35].

In the research by F. Rausche analysis results of dynamic and static load testing for end-bearing and friction piles was performed. Conclusions of the research are similar to others, and they are:

- dynamic and static load tests should be performed at the same time to neglect differences in soil condition caused by time;
- if static load test does not reach the failure criteria, the results can be extrapolated;
- if impact hammer cannot provide sufficient impact energy then correlation between load test may not be possible;
- for different load tests different failure criteria should be established [36].

4.2. Suggested testing approach

In given thesis combination of dynamic load (pile driving formula) and static load testing is suggested as possible solution because of several advantages of pile driving formulas comparing to any other dynamic load test. These advantages are:

- only measurement of permanent set per blow is required, excluding measurements of acceleration, strain and impact energy at pile head;
- fast execution of the test comparing other dynamic load test because no specific equipment is needed to attach and calibrate;

It is important to mention, that such approach add uncertainties during the verification of pile bearing capacity due to fact that measurement of impact blow energy and strain at the pile head gives real values of transferred impact energy and elastic deformation of the pile.

In research part two pile driving formulas will be reviewed from standards EN ISO 22477-4 (Equation 4.1) and СП 24.13330 (Equation 4.2 and Equation 4.3). For both empirical correlation coefficients η will be established. Empirical factor η in both approaches are interpreted completely different, so to avoid misapprehension of factor η it will be changed to α and β in equations depending on a method.

α-approach:

$$R = \frac{\alpha E_p C_r}{s_p + s_{el}} \qquad (4.1)$$

where

R – pile bearing capacity, kN

α – empirical coefficient

E_p – impact potential energy, kJ (kNm)

C_r – impact hammer efficiency coefficient

s_p – set per blow, m

s_{el} – elastic pile deformation, m

β-approach:

$$R = \frac{\beta A}{2}\left(\sqrt{1 + \frac{4E_d}{\beta A s_p} * \frac{m_1 + \varepsilon^2(m_2 + m_3)}{m_1 + m_2 + m_3}} - 1\right) \qquad \text{if } s_p \geq 2\ \text{mm} \qquad (4.2)$$

$$R = \frac{1}{2\theta} * \frac{2s_p + s_{el}}{s_p + s_{el}}\left(\sqrt{1 + \frac{8E_d(s_p + s_{el})}{(2s_p + s_{el})^2} * \frac{m_4}{m_4 + m_2}\theta} - 1\right) \qquad \text{if } s_p < 2\ \text{mm} \qquad (4.3)$$

where

R – pile bearing capacity, kN

β_1 – empirical coefficient for set per blow less than 2 mm, kN/m^2

A – pile cross-section area, m^2

E_d – hammer design impact energy, kJ (kNm)

s_p – set per blow (permanent pile displacement per blow), m

s_{el} – elastic deformation of pile caused by impact hammer blow, m

ε – impact recovery coefficient ($\varepsilon = 0.2$)

m_1 – total weight of impact hammer, kg

m_2 – total pile weight, kg

m_3 – weight of additional equipment used (driving caps, driving extenders etc.), kg

m_4 – weight of hammer impact part, kg

β_2 – coefficient calculated by Equation (4.3) if set per blow is higher than 2 mm, 1/kN

$$\theta = \frac{1}{4}\left(\frac{n_b}{A} + \frac{n_f}{A_f}\right)\frac{m_4}{m_4 + m_2}\sqrt{2g(H - h)} \qquad (4.3)$$

where

n_b – correlation coefficient between dynamic and static impact for pile base resistance (n_b = 0.00025), s·m/kN

n_f – correlation coefficient between dynamic and static impact for pile shaft resistance (n_f = 0.025), s·m/kN

g – acceleration due to gravity (g = 9.81), m/s^2

H – hammer impact part drop height, m

h – first rebound height of diesel hammer after the blow (h = 0 for different type hammers), m

A_f – pile friction area, m^2

Comparing both approaches it can be stated that α-approach has severe disadvantage comparing to β-approach because as factor α is established to pile with specific length. Difference in length change pile mass causes difference in dynamics that is considered in α-approach but is not in β-approach. Variation of pile length is common in soil conditions where soil layers with high bearing capacity can be reached at different depth, and it is one of the main reasons why shorter piles may be considered as weak, therefore execution of a load test on shorter piles may be requested.

For both approaches it is necessary to establish elastic deformation at pile head but without specific equipment it is impossible, therefore to at least approximate pile elastic deformation Equation (4.4) can be used.

$$K_{dynamic} = \frac{\sigma_{dynamic}}{\sigma_{static}} = \frac{\varepsilon_{dynamic}}{\varepsilon_{static}} = \frac{\Delta_{dynamic}}{\Delta_{static}} \qquad (4.4)$$

where

$K_{dynamic}$ – loading dynamic coefficient

$\sigma_{dynamic}$ – stress caused by dynamic loading, N/mm^2

σ_{static} – stress caused by static impact, N/mm^2

$\varepsilon_{dynamic}$ – strain caused by dynamic impact

ε_{static} – strain caused by static loading

$\Delta_{dynamic}$ – deformation caused by dynamic loading, mm

Δ_{static} – deformation caused by static loading, mm

Dynamic coefficient can be calculated by Equation (4.5).

$$K_{dynamic} = 1 + \sqrt{1 + \frac{1 + \frac{m_1}{3m_2}}{\left(1 + \frac{m_1}{2m_2}\right)^2} * \frac{2h}{\Delta_{static}}} \qquad (4.5)$$

where

m_1 – pile mass, kg

m_2 – hammer impact part mass, kg

h – impact hammer drop height, m

Δ_{static} – deformation caused by static loading calculated by Equation (4.6), m

$$\Delta_{static} = \mu \frac{m_2 g l}{AE} \qquad (4.6)$$

where

μ – coefficient depending on type of pile (1 for end-bearing piles, 0.5 for friction piles)

g – acceleration due to gravity (g = 9.81), m/s^2

l – pile length, m

A – pile cross-section area, m^2

E – pile material modulus of elasticity, N/m^2

Coefficient μ in Equation (4.6) is dependent on distribution of resistance through soil layers and shape of axial force diagram. Roughly it can be assumed that end-bearing pile will have rectangular axial force diagram, therefore coefficient μ is equal to 1. For friction piles axial force diagram will have triangular form (half of rectangle), therefore coefficient μ is equal to 0.5.

5. Combining static load test with pile driving formulas

5.1. Introduction

To combine testing approaches it is necessary to have static load testing results and pile driving records consisting of pile geometrical parameters, hammer type, impact energy and measurements of set per blow. For given purpose static load testing results of 6 open-end steel piles and corresponding driving records are used.

During the construction of harbour structure open-end steel tubular piles were driven in soil using diesel impact hammer Delmag D46-32 with maximum impact energy of 144 kNm. All piles have length of 23.4 m and tubular cross-section with diameter 1016 mm and wall thickness 13 mm. Design impact energy is determined according to Russian standard СП 24.13330 assuming that diesel hammer reaches 90% of its maximum impact energy during record of set per blow. Ram weight of the hammer is 4.6 t, stroke height 3.2 m. Weight of the steel pile is 7524.5 kg. General parameters necessary for following calculations are given in Table 5.1.

Table 5.1. General parameters from pile driving records

No.	Pile No.	Pile type	Cross-section area, m^2	Pile length, m	Impact hammer	Maximum impact energy, kNm	Design impact energy, kNm	Set per blow, mm
1	5	Ø1016x13 mm steel pile	0.0410	23.40	Delmag D46-32 (diesel)	144.40	129.96	0.170
2	12	Ø1016x13 mm steel pile	0.0410	23.40	Delmag D46-32 (diesel)	144.40	129.96	0.700
3	13	Ø1016x13 mm steel pile	0.0410	23.40	Delmag D46-32 (diesel)	144.40	129.96	0.800
4	15	Ø1016x13 mm steel pile	0.0410	23.40	Delmag D46-32 (diesel)	144.40	129.96	0.300
5	16	Ø1016x13 mm steel pile	0.0410	23.40	Delmag D46-32 (diesel)	144.40	129.96	1.397
6	19	Ø1016x13 mm steel pile	0.0410	23.40	Delmag D46-32 (diesel)	144.40	129.96	1.149

5.2. Geotechnical conditions

Geotechnical conditions of construction site consist of mixture of coarse and fine soils. Sand mixtures (mostly technogenic) are located closer to the surface of existing harbour structure and fine soils bellow. From parameters given in geotechnical report several parameters indicate hard driving conditions that could require additional treatment like pre-augering or application of high yield strength steel. Given parameters are undrained shear strength (c_u) and liquidity index (I_L). Liquidity index for both soils is negative that indicates that the soil is bellow plastic limit (soil is stiff). For silty clay c_u value varies from 170 to 370 kPa (270 on average) and for clay c_u lays between 170 and 270 kPa (220 on average) that indicates hard consistency. [Terzaghi peck]

Tested piles are mostly driven into silty clay and clay stratum and given soils comparing to coarse soils have high skin friction resistance, so correct analysis can be executed only with understanding of distribution between skin friction and end-bearing resistance.

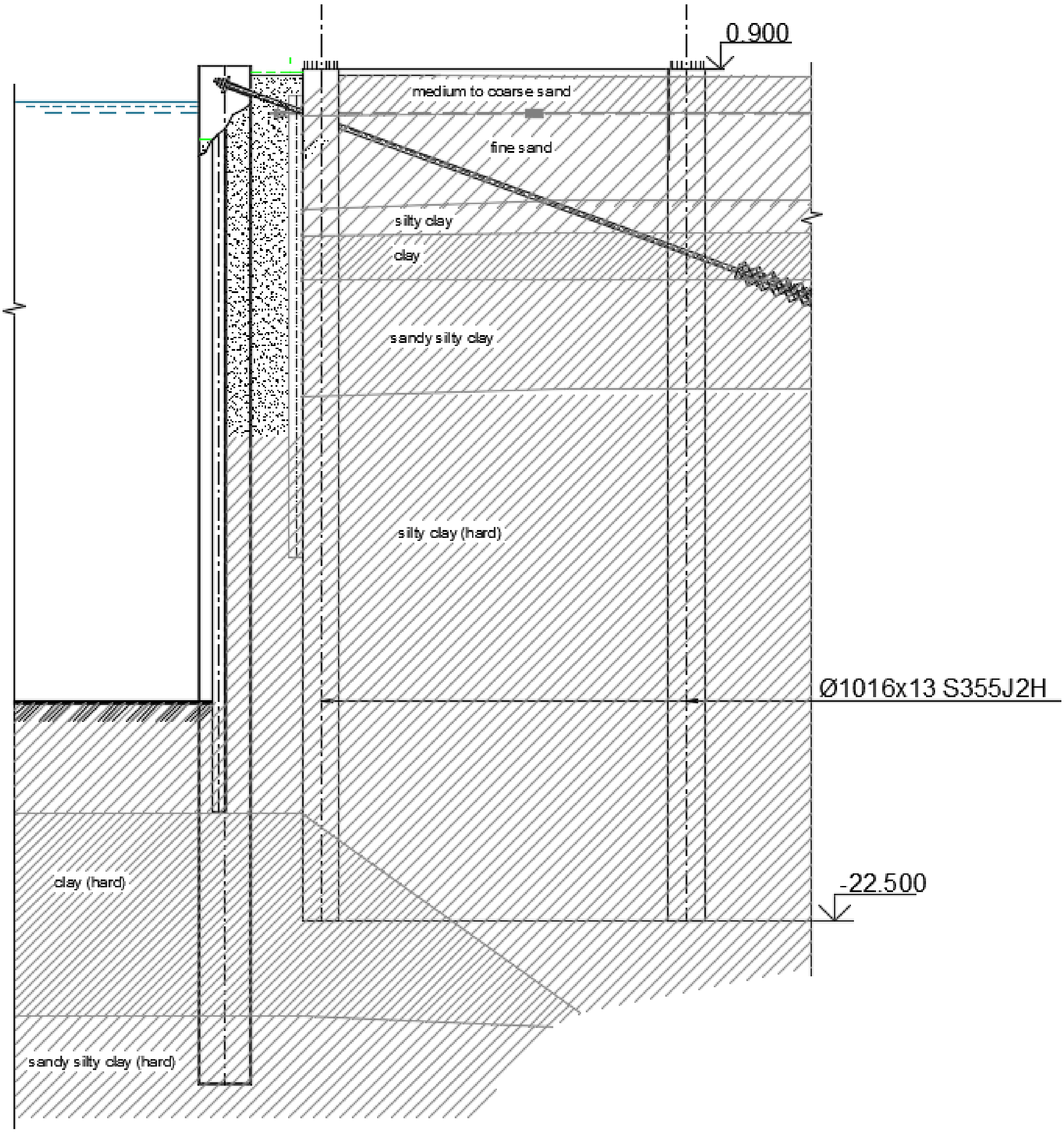

Fig.5.1. Geotechnical profile of construction site

5.3. Results of static load testing

Execution of static load testing was performed according to EN ISO 22477-1 with 8 loading steps up to the maximum test load of 3200 kN and 4 unloading steps. Each loading step was held for 1 hour to determine pile behaviour under the load in the terms of creep. Load-displacement curves as result of testing with static load is given bellow.

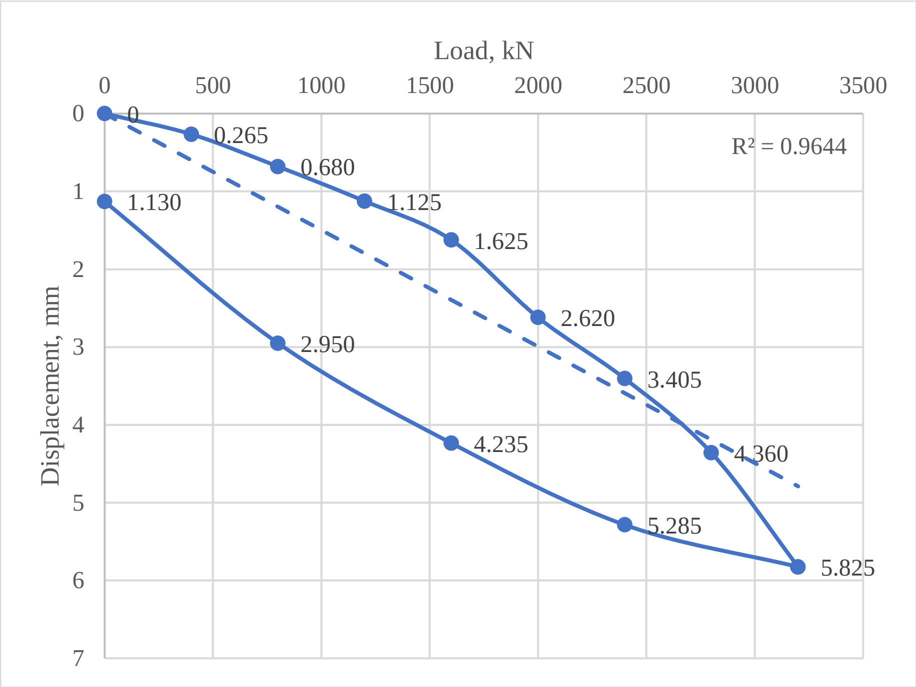

Fig.5.2. Pile No.5 load-displacement curve

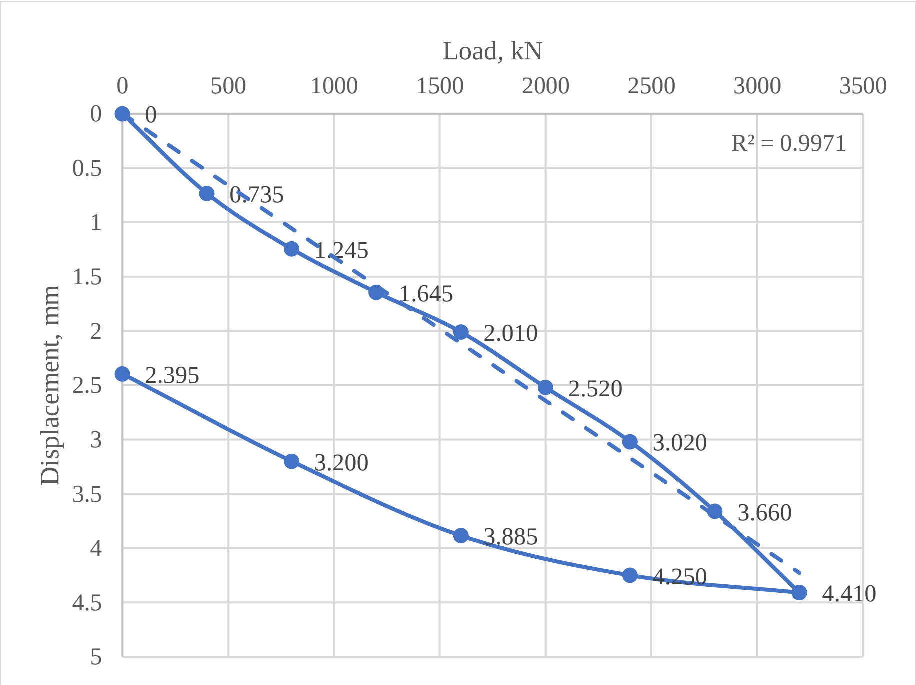

Fig.5.3. Pile No.12 load-displacement curve

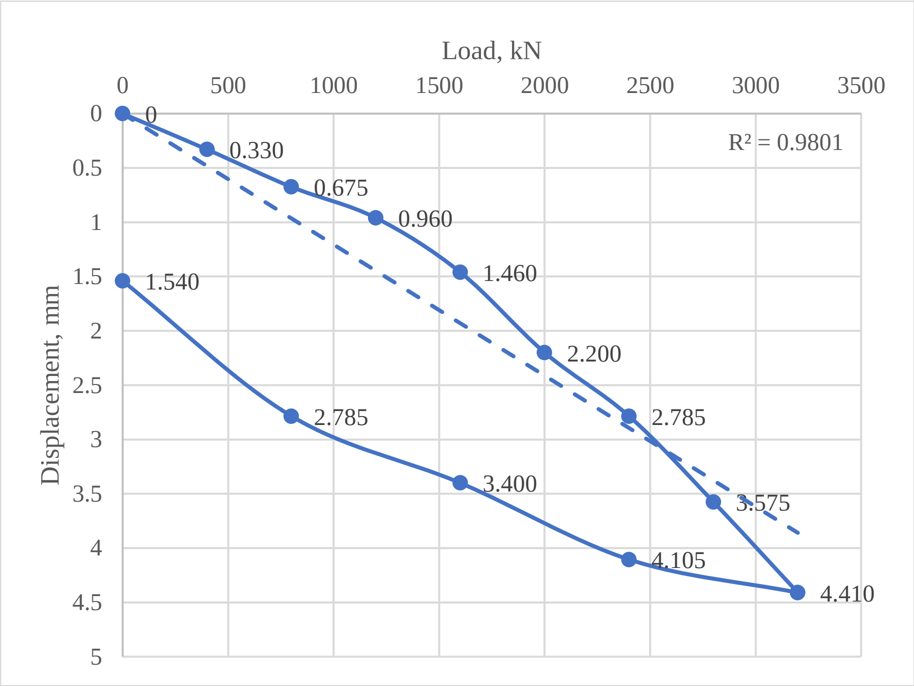

Fig.5.4. Pile No.13 load-displacement curve

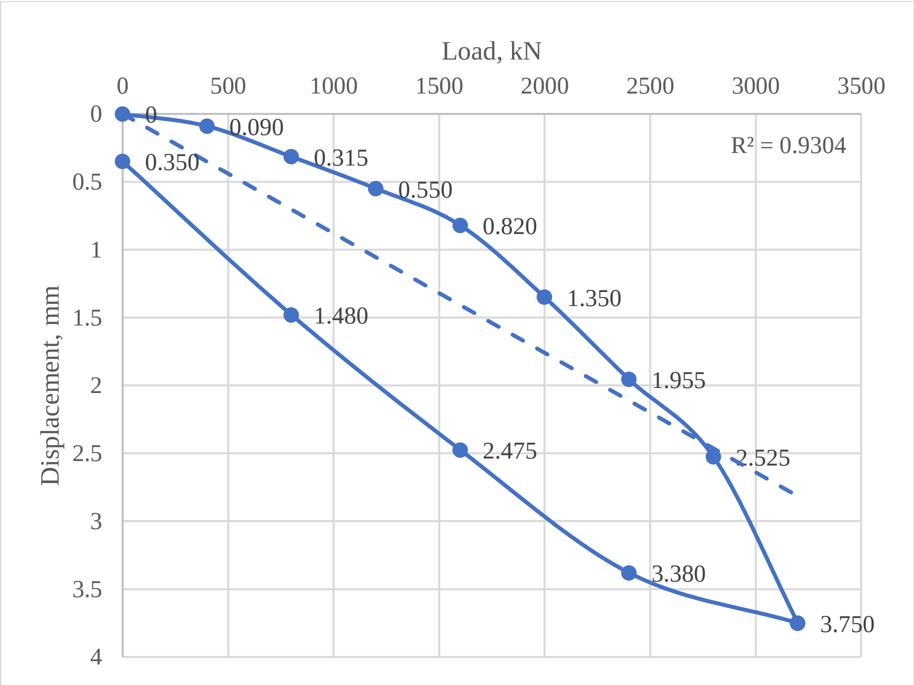

Fig.5.5. Pile No.15 load-displacement curve

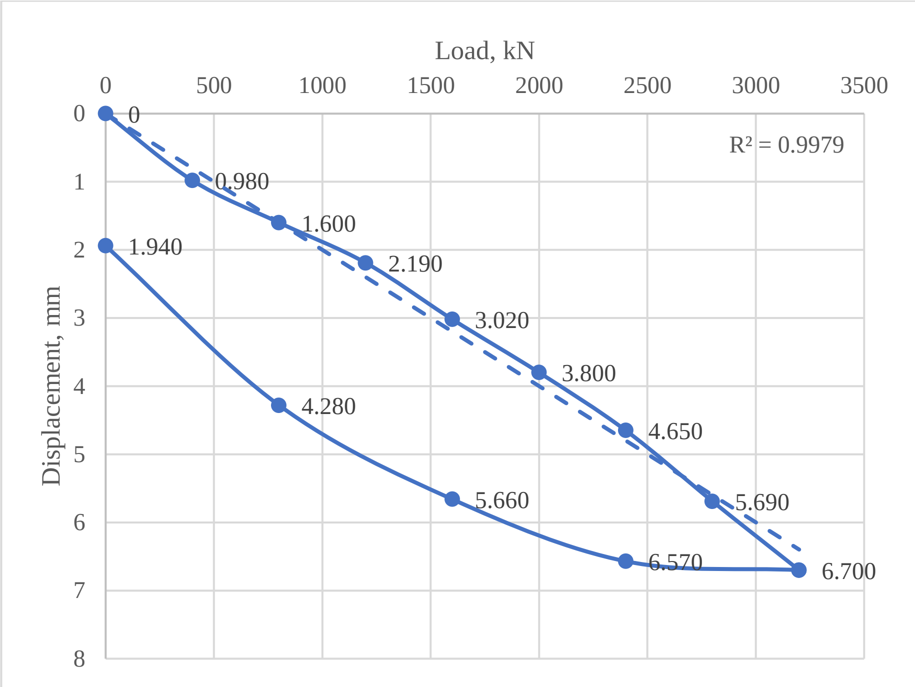

Fig.5.6. Pile No.16 load-displacement curve

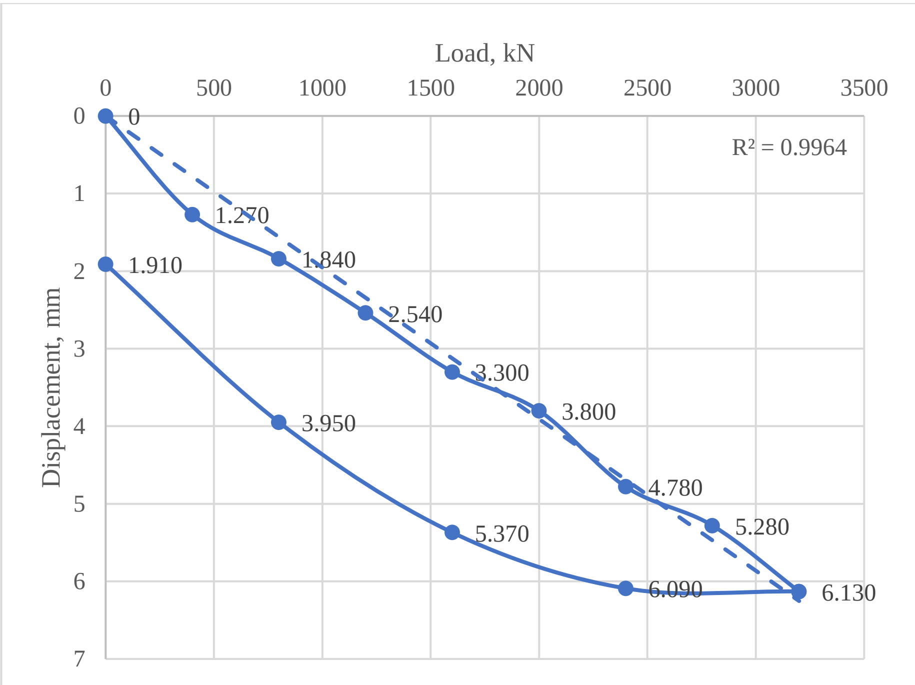

Fig.5.7. Pile No.19 load-displacement curve

Given load-displacement curves can be evaluated according to chapter 3.2.2. As it was mentioned before, main criteria describing pile load bearing capacity is displacement under the load and shape of load-displacement curve. Limiting value of displacement under the load is 10% of pile base diameter, therefore displacement should not exceed 101.6 mm and as it is seen in load-displacement curves displacement does not exceed 6.7 mm. The shape of load-displacement curve can be divided in 3 sections – first section obeys the Hooks law (displacement is proportional to load), second section starts with yield point and after this point increasement in load causes higher deformations, therefore displacement is not proportional to load, and third section where displacement increases without increase of force.

As it is seen load-displacement curves of tested piles approximation line of loading sequence lays close to measured data with lowest R^2 value of 0.9304 for pile No.15. In addition, it is seen that permanent displacement of piles does not exceed 2.395 mm, so it can be stated that test load of 3200 kN does not reach the yield point of pile bearing capacity, therefore actual pile bearing capacity is higher than a test load.

5.4. *α*-approach

To establish coefficient α Equation (5.1) should be used. In given expression value s_{el} is calculated theoretically due to lack of real measurements by Equation (5.2). Also, it should be mentioned that blow efficiency of 90% is given by construction code and can cause uncertainty.

$$\alpha = R_{stat}\frac{s_{el} + s_p}{E_d} \qquad (5.1)$$

where

R_{stat} – pile bearing capacity proven by static load testing, kN

s_{el} – elastic set per blow determined by Equation (5.2), m

s_p – permanent set per blow taken from Table 5.1., m

E_d – design impact energy taken from Table 5.1., kNm

$$s_{el} = \Delta_{static}K_{dynamic} \qquad (5.2)$$

where

Δ_{static} – value determined bellow by Equation (4.6) assuming pile is end-bearing, m

$k_{dynamic}$ – coefficient determined bellow by Equation (4.5)

$$\Delta_{static} = \frac{4600 * 9.81 * 3.2}{0.0410 * 210 * 10^9} = 0.00001679 = 1.679 * 10^{-5}\ (m)$$

$$K_{dynamic} = 1 + \sqrt{1 + \frac{1 + \frac{7524.5}{3 * 4600}}{\left(1 + \frac{7524.5}{2 * 4600}\right)^2} * \frac{2 * 3.2}{1.679 * 10^{-5}}} = 423.225$$

$$s_{el} = 1.679 * 10^{-5} * 423.225 = 0.00711\ (m)$$

Results of calculated α coefficient for each pile is given in Table 5.2.

Table 5.2. α coefficient for each pile

No.	Pile number	s_p, mm	s_{el}, mm	E_d, kNm	R_{stat}, kN	α
1	5	0.170	7.105	129.96	3200	0.179
2	12	0.700	7.105	129.96	3200	0.192
3	13	0.800	7.105	129.96	3200	0.195
4	15	0.300	7.105	129.96	3200	0.182
5	16	1.397	7.105	129.96	3200	0.209
6	19	1.149	7.105	129.96	3200	0.203

As it seen in the results α coefficient varies from 0.179 to 0.209. Coefficient is smaller for piles with lower set per blow and otherwards for piles with higher permanent set. For further application two approaches are suggested:

- application of correlation factors dependent on test number to mean and minimum value of established coefficients similarly to approach of Eurocode 7;
- using minimum value reduced by safety factor for more conservative approach.

Considering theoretical value of elastic set without recording real measurements assumes that all piles are in the same geotechnical conditions (resistance distributes through length equally for all piles) that creates uncertainty that should be considered with application of higher safety or model correction factors.

5.5. β-approach

As it is seen in Equations (5.3) and (5.4) given approach divides in two conditions depending on permanent set per blow. Driving record of piles shows that permanent set per blow for all piles is lower than 2 mm, therefore application of given method and comparison with α-approach using test data is not completely correct. For calculation of pile bearing capacity using given approach, it is necessary to establish coefficient θ that includes factors – n_b and n_f. These factors are empirical coefficients for both pile base and shaft resistances. These parameters cannot be established without deriving pile bearing capacity into base and shaft resistances.

$$R = \frac{\beta A}{2}\left(\sqrt{1 + \frac{4E_d}{\beta A s_p} * \frac{m_1 + \varepsilon^2(m_2 + m_3)}{m_1 + m_2 + m_3}} - 1\right) \qquad \text{if } s_p \geq 2 \text{ mm} \qquad (5.3)$$

$$R = \frac{1}{2\theta} * \frac{2s_p + s_{el}}{s_p + s_{el}}\left(\sqrt{1 + \frac{8E_d(s_p + s_{el})}{(2s_p + s_{el})^2} * \frac{m_4}{m_4 + m_2}\theta} - 1\right) \qquad \text{if } s_p < 2 \text{ mm} \qquad (5.4)$$

First, to establish β value it should be expressed from Equation (5.3), final simplification is expressed in Equation (5.5).

$$\beta = \frac{{R_{stat}}^2 s_p}{A\left(E_d\left(\frac{m_1 + \varepsilon^2(m_2 + m_3)}{m_1 + m_2 + m_3}\right) - R_{stat} s_p\right)} \qquad (5.5)$$

Similarly, to previous approach β value is dependent on pile bearing capacity established by static load test, and as before it is 3200 kN. β values for all piles are given in Table 5.3.

Table 5.3. β coefficient for each pile

No.	Pile number	R_{stat}, kN	s_p, mm	E_d, kNm	m_1, kg	m_2, kg	m_3, kg	ε	A, m^2	β, kN/m^2
1	5	3200	0.170	129.96	9300	7524.54	500	0.2	0.0410	593.29
2	12	3200	0.700	129.96	9300	7524.54	500	0.2	0.0410	2502.18
3	13	3200	0.800	129.96	9300	7524.54	500	0.2	0.0410	2872.78
4	15	3200	0.300	129.96	9300	7524.54	500	0.2	0.0410	1053.09
5	16	3200	1.397	129.96	9300	7524.54	500	0.2	0.0410	5158.15
6	19	3200	1.149	129.96	9300	7524.54	500	0.2	0.0410	4193.31

Value of β coefficient varies from 593.29 kN/m^2 to 5158.15 kN/m^2 and as in previous approach values are higher if set per blow is higher. Comparing to previous approach for further applications two options can be considered – application of correlation factors depending on tests performed on mean and minimum value or using minimum value reduced by safety factor.

5.6. Comparison

Comparison between the approaches is performed by calculating pile bearing capacity for different set per blow values. For comparison mean values of coefficient α and β are taken without application of any correlation or safety factors. Results are compiled in Table 5.4. and in Fig.5.8.

Table 5.4. Comparison of α and β approach results

s_p, mm	E_d, kNm	α approach			β approach						
		α_{mean}	s_{el}, mm	R_{dyn}, kN	β_{mean}, kN/m^2	A, m^2	m_1, kg	m_2, kg	m_3, kg	ε	R_{dyn}, kN
0.20	129.96	0.193	7.105	3442.12	2728.80	0.0410	9300	7524.54	500	0.2	6295.57
0.40	129.96	0.193	7.105	3350.38	2728.80	0.0410	9300	7524.54	500	0.2	4435.44
0.60	129.96	0.193	7.105	3263.41	2728.80	0.0410	9300	7524.54	500	0.2	3611.41
0.80	129.96	0.193	7.105	3180.84	2728.80	0.0410	9300	7524.54	500	0.2	3120.21
1.00	129.96	0.193	7.105	3102.34	2728.80	0.0410	9300	7524.54	500	0.2	2785.01
1.20	129.96	0.193	7.105	3027.63	2728.80	0.0410	9300	7524.54	500	0.2	2537.58
1.40	129.96	0.193	7.105	2956.43	2728.80	0.0410	9300	7524.54	500	0.2	2345.29
1.60	129.96	0.193	7.105	2888.50	2728.80	0.0410	9300	7524.54	500	0.2	2190.30
1.80	129.96	0.193	7.105	2823.62	2728.80	0.0410	9300	7524.54	500	0.2	2061.92
2.00	129.96	0.193	7.105	2761.59	2728.80	0.0410	9300	7524.54	500	0.2	1953.32
2.20	129.96	0.193	7.105	2702.23	2728.80	0.0410	9300	7524.54	500	0.2	1859.89
2.40	129.96	0.193	7.105	2645.37	2728.80	0.0410	9300	7524.54	500	0.2	1778.40
2.60	129.96	0.193	7.105	2590.85	2728.80	0.0410	9300	7524.54	500	0.2	1706.51
2.80	129.96	0.193	7.105	2538.54	2728.80	0.0410	9300	7524.54	500	0.2	1642.46
3.00	129.96	0.193	7.105	2488.29	2728.80	0.0410	9300	7524.54	500	0.2	1584.94
3.20	129.96	0.193	7.105	2440.00	2728.80	0.0410	9300	7524.54	500	0.2	1532.90
3.40	129.96	0.193	7.105	2393.54	2728.80	0.0410	9300	7524.54	500	0.2	1485.52
3.60	129.96	0.193	7.105	2348.82	2728.80	0.0410	9300	7524.54	500	0.2	1442.15
3.80	129.96	0.193	7.105	2305.74	2728.80	0.0410	9300	7524.54	500	0.2	1402.25
4.00	129.96	0.193	7.105	2264.21	2728.80	0.0410	9300	7524.54	500	0.2	1365.38
4.20	129.96	0.193	7.105	2224.15	2728.80	0.0410	9300	7524.54	500	0.2	1331.18
4.40	129.96	0.193	7.105	2185.49	2728.80	0.0410	9300	7524.54	500	0.2	1299.35
4.60	129.96	0.193	7.105	2148.14	2728.80	0.0410	9300	7524.54	500	0.2	1269.61
4.80	129.96	0.193	7.105	2112.05	2728.80	0.0410	9300	7524.54	500	0.2	1241.75
5.00	129.96	0.193	7.105	2077.16	2728.80	0.0410	9300	7524.54	500	0.2	1215.58

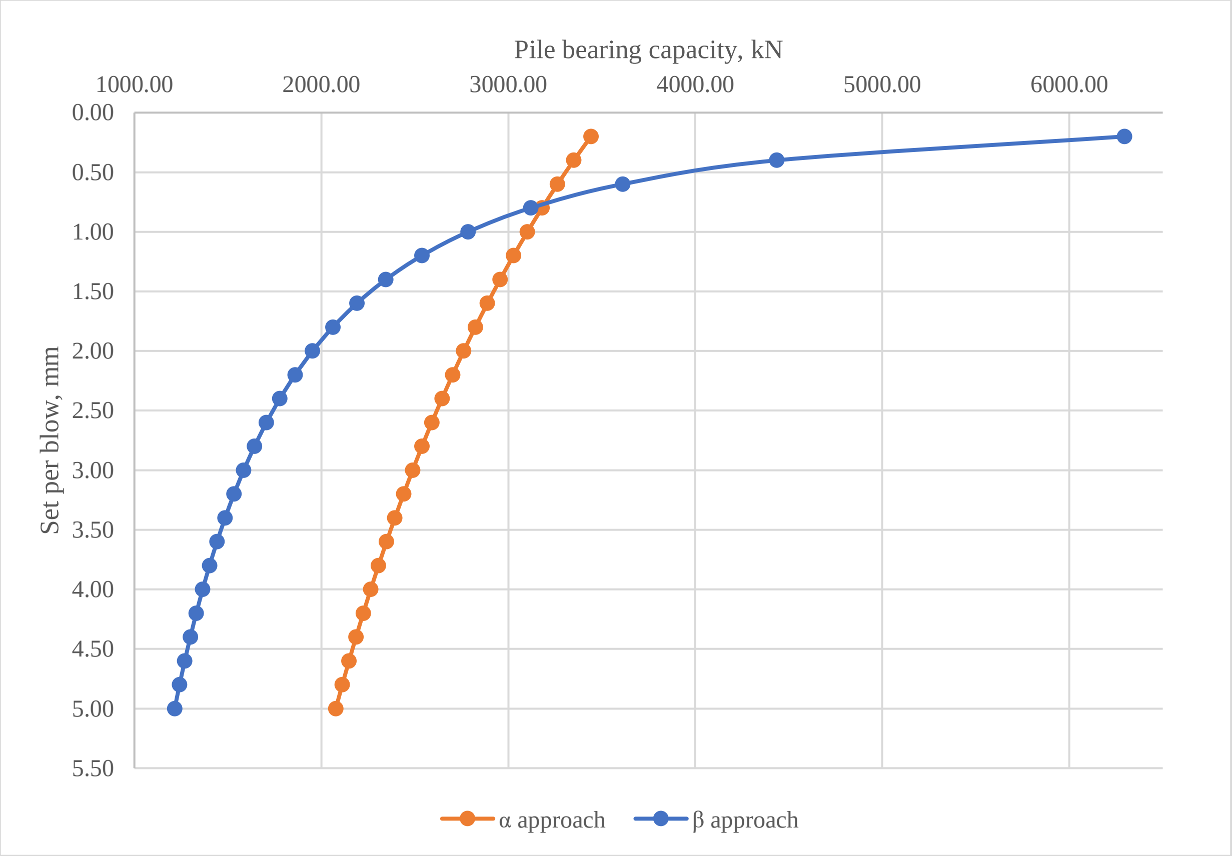

Fig.5.8. Comparison of α and β approaches

It is clearly seen that pile bearing capacity in β-approach rapidly increases when set per blow is lower than 2 mm, therefore it is clear why given approach is divided in two formulas depending on permanent set per blow. For the set per blow values lower than 0.75 mm β-approach gives significantly higher pile bearing capacity values than α-approach and given values most likely are higher than actual pile bearing capacity.

On the other side α-approach in general gives higher pile bearing capacity values and connection between set per blow and pile bearing capacity is close to linear.

6. Conclusions

In general, it can be stated than goals set in introduction are reached – different theoretical and practical load bearing capacity establishment methods for driven piles are compiled and pile bearing capacity analysis using different pile driving formulas is performed. Suggestions on rational and cost-efficient testing approach, shortcomings of research and proposals for further research is given bellow.

Suggestions on rational and cost-efficient testing approach:

- firstly, each test program shall fulfil requirements of Eurocode 7 and other construction codes in the terms of minimum tests performed, therefore static load test is a necessity for both - establishment of correlation coefficients α or β for specific site and a condition for meeting the standard requirements;
- secondly, correlation between static load testing and dynamic load testing should be done only for same material, length and cross-section piles driven in the same geotechnical conditions with the same pile driving equipment;
- thirdly, application of correlation and safety factors depending on number of load tests performed shall be considered to establish the balance between measured pile bearing capacity reduction to design value and number of tests executed;
- lastly, from the comparison results it can be said than β-approach gives more conservative values of pile bearing capacity if permanent set per blow is higher than 2 mm and significant increase in pile bearing capacity is noted if given value is lower than 2 mm, therefore β-approach shall be limited to piles with permanent set per blow higher than 2 mm.

Shortcomings and difficulties encountered during research part are related to uncertainties caused by lack of several measurements that require specific equipment. Factors that affected research are:

- static load testing was performed up to load of 3200 kN, which was not the pile ultimate bearing capacity, therefore correlation factors are established from value that does not reach soil resistance;
- no measurements of elastic set per blow are made, so measured values were replaced by theoretical values assuming that pile is completely end-bearing excluding friction resistance;

- design impact energy was derived from impact hammer manufacturers data sheet assuming 90% efficiency and given assumption does not include impact energy loses in pile driving cap, general condition of impact hammer and any other factors that may cause uncertainties;
- for proper β-approach analysis end-bearing and skin friction resistances should be differentiated and only possible way to do so is derive skin friction from established total pile bearing and pile end-bearing capacity measured with special measuring devices attached to pile end.

Following proposals for further research related to shortcomings described before are:

- static load test should be executed to establish pile ultimate bearing capacity and test load should be limited to pile cross-sectional resistance;
- measurements of elastic set per blow and actual impact energy should be made;
- hydraulic impact hammer is suggested due to fact that hydraulic impact hammers provide constant impact energy unlike diesel impact hammers, which impact energy is dependent on rebound;
- for further research on β-method test piles should be equipped with pile end-bearing measurement devices.

7. References

1. Tomlinson, M., Woodward, J. *Pile Design and Construction Practice. Sixth Edition*. Boca Raton: CRC Press Taylor & Francis Group, 2015. 574 p. ISBN-13: 978-1-4665-9264-3.

2. *EN 12699 "Execution of special geotechnical work – Displacement piles"*. European Committee for Standardization. Brussels, 2000. 46 p.

3. ArcelorMittal. *Piling Handbook. 9th edition*. Luxembourg: ArcelorMittal Commercial RPS, 2016. 455 p. ISBN: 978-99959-0-194-3.

4. Fleming, K., Weltman, A., Randolph, M., Elson, K. *Piling Engineering. Third Edition*. Abington: Taylor & Francis Group, 2009. 398 p. ISBN: 0-203-93764-3.

5. *EN 1536 "Execution of special geotechnical work – Bored piles"*. European Committee for Standardization. Brussels, 2015. 82 p.

6. BAUER Maschinen GmbH. *Kellybohrverfahren* [online]. bauer.de [viewed 26 December 2021]. Available from: *https://www.bauer.de/bma/Verfahrensuebersicht/Bohrverfahren/Kellybohrverfahren/*

7. *EN 14199* "*Execution of special geotechnical work – Micropiles*" European Committee for Standardization. Brussels, 2015. 62 p.

8. Friedr. Ischebeck GmbH. *One installation method for all applications* [online]. ischebeck.de [viewed 26 December 2021]. Available from: *https://www.ischebeck.de/en//product/geotechnical-systems/one-installation-method/*

9. *EN 1997-1 "Eurocode 7: Geotechnical design – Part 1: General rules"*. European Committee for Standardization. Brussels, 2004. 171 p.

10. Bowles, J. E. *Foundation Analysis and Design. Fifth Edition*. Singapore: McGraw-Hill Companies, 1997. 1230 p. ISBN: 0-07-912247-7.

11. *EN 1997-2 "Eurocode 7: Geotechnical design – Part 2: Ground investigation and testing"*. European Committee for Standardization. Brussels, 2007. 199 p.

12. *Electric Cone Penetration Test (CPT)* [online]. soiltestingequipment.blogspot.com [viewed 26 December 2021]. Available from: *http://soiltestingequipment.blogspot.com/2010/01/electric-cone-penetration-test-cpt.html*

13. Jacobs, P. *Cone Penetration Testing (CPT). "Simplified Description of the Use and Design Methods for CPTs in Ground Engineering"*. Wallingford: Fugro Engineering Services Ltd, 2004. 50 p.

14. Titi, H. H., Abu-Farsakh, M. Y. *Evaluation of Bearing Capacity of Piles from Cone Penetration Test Data*. Baton Rouge: Louisiana Transportation Research Center, 1999. 115 p.

15. Robertson, P. K., Cabal, K. L (May 2010). *Estimating soil unit weight from CPT*. 2nd International Symposium on Cone Penetration Testing Huntington Beach, CA, USA.
16. Larsson, R. *Investigation and load tests in silty soils. Results from a series of investigations in silty soils in Sweden*. Linkoping: Swedish Geotechnical Institute, 1997. 266 p. ISSN: 0348-0755.
17. Committee for Waterfront Structures of the German Port Technology Association and the German Geotechnical Society. *Recommendations of the Committee for Waterfront Structures Harbours and Waterways EAU 2012. 9th Edition*. Berlin: Wilhelm Ernst & Sohn, 2015. 683 p. ISBN: 978-3-433-03110-0.
18. *NEN 6743-1 "Geotechnics – Calculation method for bearing capacity of pile foundation – Compression piles"*. Royal Netherlands Standardization Institute. Delft, 2006. 45 p.
19. *Noteikumi par Latvijas būvnormatīvu LBN 207-15 "Ģeotehniskā projektēšana"* (2015) [online]. Republic of Latvia Cabinet Regulation No. 265, adopted on 2 June 2015, Riga, likumi.lv [viewed 26 December 2021]. Available from: https://likumi.lv/ta/id/274440-noteikumi-par-latvijas-buvnormativu-lbn-207-15-geotehniska-projektesana
20. *EN ISO 22477-1 "Geotechnical investigation and testing – Testing of geotechnical structures – Part 1: Testing of piles: static compression load testing"*. European Committee for Standardization. Brussels, 2018. 31 p.
21. German Geotechnical Society. *Recommendations on Piling (EA-Pfahle)*. Berlin: Wilhelm Ernst & Sohn, 2014. 494 p. ISBN: 978-3-433-03018-9.
22. Likins, G., Rausche, F., Goble, G. G. (2000). *High strain dynamic pile testing, equipment and practice*. 6th International Conference on Application of Stress-Wave Theory to Piles. Quality Assurance on Land and Offshore Piling.
23. *EN ISO 22477-4 "Geotechnical investigation and testing – Testing of geotechnical structures – Part 4: Testing of piles: dynamic load testing"*. European Committee for Standardization. Brussels, 2018. 61 p.
24. Juntann Oy. *Junttan Hydraulic Impact Hammer* [online]. junttan.com [viewed 26 December 2021]. Available from: https://junttan.com/product/hhks-series/
25. Allin, R., Likins, G., Honeycutt, J. (17-21 March 2015). *Pile Driving Formulas Revisited*. International Foundations Congress and Equipment Expo 2015, San Antonio, Texas, USA.
26. Dawson Construction Plant Ltd. *Dawson HPH2400 Hydraulic Hammer – Energy Monitoring and Ultimate Pile Bearing Capacity Chart* [online]. dcpuk.com [viewed 27 December 2021]. Available from:

http://wequips.com/img/fotos/dawson/HPH2400_Bearing_Capacity_Chart_-_metric_version.pdf
27. Sorensen, T., Hansen, J. B. (1957). Pile driving formulae, an investigation based on dimensional considerations and a statistical analysis. *Proceedings of the 4th International Conference on Soil Mechanics and Foundation Engineering*, pp. 61-65, London.
28. *СП 24.13330.2011 "СВОД ПРАВИЛ – СВАЙНЫЕ ФУНДАМЕНТЫ"*. Federal Agency for Technical Regulation and Metrology. Moscow, 2011.
29. Holscher, P. van Tol, F. *Rapid Load Testing on Piles*. London: Taylor & Francis Group, 2009. 192 p. ISBN: 978-0-415-48297-4.
30. Matsumoto, T. (2019). Rapid Load Test of Piles. *IPA News Letter*, Vol. 4, Issue 2 June 2019.
31. *EN ISO 22477-10 "Geotechnical investigation and testing – Testing of geotechnical structures – Part 10: Testing of piles: rapid load testing"*. European Committee for Standardization. Brussels, 2016. 32 p.
32. Brown, M. (2016). Design methods based upon rapid load tests. *ISSMGE – ETC 3 International Symposium on Design of Piles in Europe*, 28-29 April 2016, Leuven, Belgium.
33. Rajagopal, C., Solanki, C. H., Tandel, Y. K. (2012). Comparison of Static and Dynamic Load Test of Pile. *The Electronic Journal of Geotechnical Engineering, 17,* 1905-1914.
34. Phetteplace, B., Hyatt, T., Alvarez, C. (2019, October). *Dynamic and Static Load Testing – A Cost Savings Approach*. 44th Annual Conference on Deep Foundations, Chicago, 2019.
35. Barbosa, P., Geduhn, M., Jardine, R. J., Schroeder, F. C. (2017). Large Scale Offshore Static Pile Tests – Practicality and Benefits. *Offshore Site Investigation and Geotechnics,* 644-651.
36. Rausche, F. (2018). *Combining Static and Dynamic Loading Test Results of Piles*. Symposium: 10th Int. Conf. on Stress Wave Theory and Testing of Deep Foundations, San Diego, USA, 2018.