Saimaa University of Applied Sciences
Faculty of Technology, Lappeenranta
Double Degree Programme in Civil and Construction Engineering

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DESIGN OF CONTINUOUS FLIGHT AUGER PILE FOUNDATION FOR A MULTI-STOREY APARTMENT BUILDING

Bachelor’s Thesis 2010

ABSTRACT

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Design of Continuous Flight Auger Pile Foundation for a Multi-Storey Apartment Building, 43 pages, 10 appendices
Saimaa University of Applied Sciences, Lappeenranta
Double Degree Programme in Civil and Construction Engineering
Bachelor’s Thesis 2010
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The object of this work was studying, systemizing and describing continuous flight auger (CFA) pile foundation design for a residential complex and calculating the bearing capacity of a single CFA pile on the basis of Russian Building Code. This thesis was written on the request of the YIT Lentek JSK Design Department.

The information for the theoretical part was gathered from Russian Building Codes, theoretical manuals, other common available materials (including the Internet), and project materials and documents kept in the archive of YIT Lentek JSC. The practical part of the study presents the calculation of bearing capacity of a single CFA pile based on Russian Building Code, called SNiP 2.02.03-85 “Pile Foundations”.

General systematization of the process of CFA pile foundation design for multi-storey apartment buildings, which is useful for further studies, was implemented on the basis of gathered and analyzed materials. Results of a single CFA pile bearing capacity calculation serves for the company as a starting point in determination of bearing capacity of a single CFA pile of a given diameter and length for further considerations and comparisons with other variants of piling.

Keywords: Continuous Flight Auger (CFA) Pile, Design, Foundation, Multi-Storey Apartment Building

CONTENTS

1 INTRODUCTION

This Bachelor’s Thesis presents research work combined with practical calculations intended to fulfill the requirements of the Double Degree Programme in Civil and Construction Engineering at Saimaa University of Applied Sciences. The focus of the research is on studying, systemizing and describing the continuous flight auger (CFA) pile foundation design for a residential complex, and the calculations consist of determination of the bearing capacity of a single CFA pile on the basis of Russian Building Norms and Rules.

Pile foundations have always occupied a worthy place in engineering practice. They have come into particularly widespread use in the last 30-35 years. This is associated with an increase in the number of storeys, an expansion of the clearance dimensions of buildings and structures, and an increase in the loads taken up per unit area. They have been used for the development of sites with unfavorable geologic-engineering conditions, complex relief, high ground-water table, etc., and in many regions, up to 40-70% of buildings and structures are raised on pile foundations. Pile foundations are most widely used in America, Holland, Italy, Russia, France, and Japan. The fact that during the last 30 years almost every fourth candidate and every third doctoral dissertation, concerning the "bed and foundation” specialty has been devoted to problems of pile foundation engineering suggests a high level of experimental-theoretical research in the field of pile foundation engineering in Russia. (Bartolomei, 1995).

The design of a pile foundation involves solving the complex problem of transferring loads from the structure through the piles to the underlying soil. Nowadays pile foundations are extensively used in Saint-Petersburg to support multi-storey apartment buildings for effective structural loads transfer from the superstructure to the ground and for avoiding the excessive settlement or lateral movement.

The study of the process of pile foundation design for this type of structure was carried out through exploration and analyses of data from the YIT Lentek JSC building pile foundation design. The considered structure is called “Vita Nova” and information about this building is also presented in the given work. Project materials and documentation, kept in the YIT Lentek JSC archive, as well as theoretical manuals and other research works were used for summarizing the theoretical information.

The calculation of the bearing capacity of a single CFA pile is implemented on the basis of SNIP 2.02.03-85 “Pile Foundations”, which is Russian Building Code. This calculation is done to get the opportunity for approximate evaluation of a single CFA pile bearing capacity for further consideration and comparison with other variants of piling.

The collection of papers appended to this thesis forms an integral part of the research and should be read in conjunction with the main text.

2 CONTINUOUS FLIGHT AUGER PILES

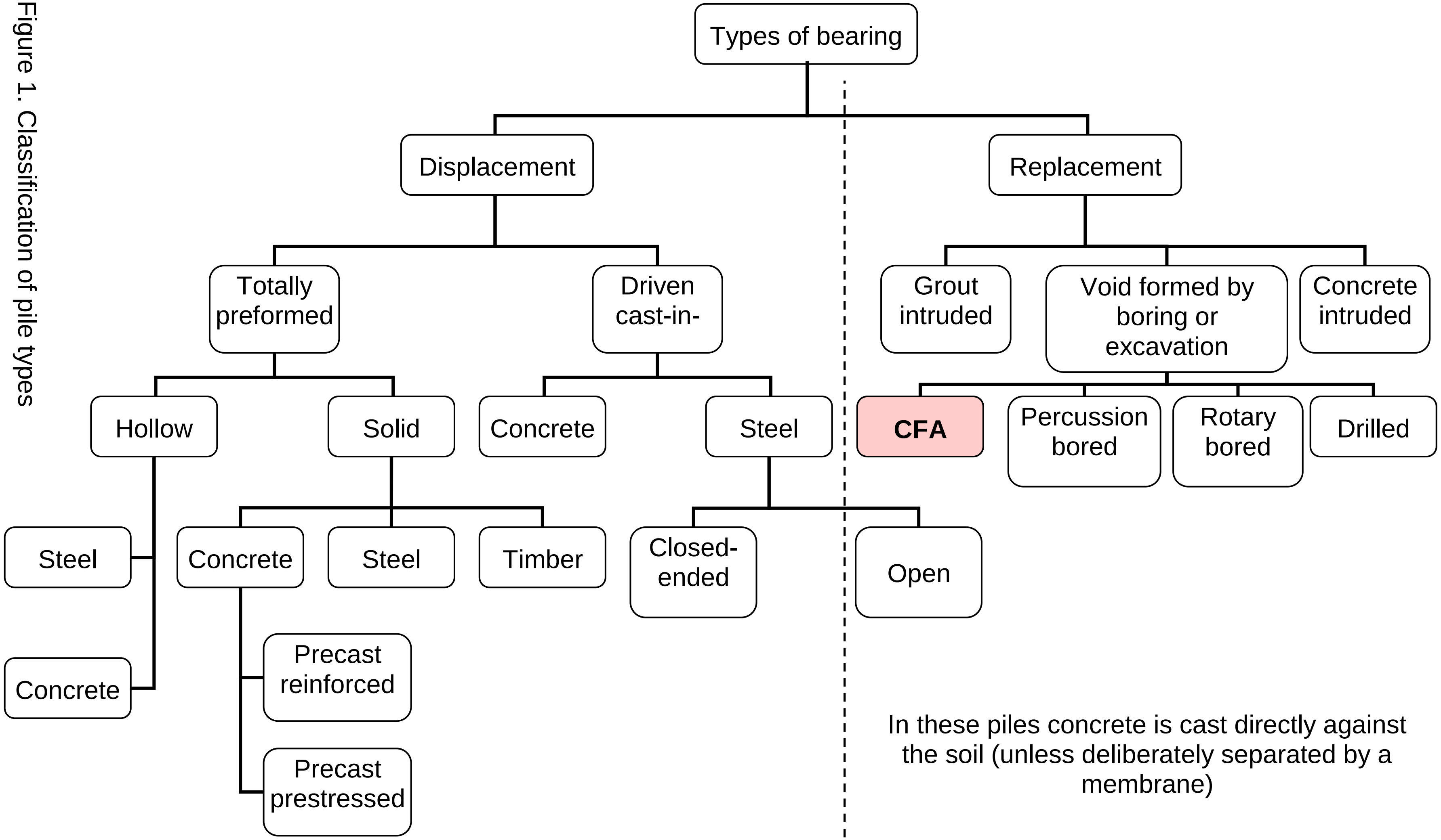
This chapter provides general information about continuous flight auger (CFA) piles which are under consideration in the course of this thesis. It is important for me to understand the nature of this pile type in general and the process of their implementation in particular, not only because it affects the calculation of pile bearing capacity (this will be mentioned in Chapter 6). A thorough understanding of construction technology of any projected structure is obligatory for the designers, and as my purpose is to get closer to their work by studying and highlighting the CFA pile foundation design, this understanding is also necessary for me.

Piles have been used to provide foundations for over 4000 years and were widely used by the Ancient Greeks and Romans. Modern piling techniques are diverse and the term “pile” is used to describe a wide variety of columnar elements in a foundation which transfers load from the superstructure through weak or compressible strata to more competent soils or rock.

Piles may be crudely classified as either displacement or replacement types, according to their method of construction. Each of these major categories can then be further divided. Figure 1 shows a classification of pile types (Baxter, 2009).

Replacement piles remove soil from the pile location to allow construction of the pile. Construction of replacement piles reduces lateral stress in the ground. The problems of noise and vibration associated with displacement piles may be avoided by using replacement piles, although this is at the cost of the loss of the benefit from soil compaction (Baxter, 2009).

Figure 1. Classification of pile types



As it is shown in Figure 1, CFA piles are subsets of bored replacement piles formed by drilling an auger with continuous helical flights into the ground to the required depth and pumping concrete through the hollow auger stem as it is subsequently withdrawn (Baxter, 2009). Since their worldwide introduction, they have become increasingly popular, as they can be considerably cheaper than alternative pile types. With proper planning and design, efficient equipment and experienced personnel, high production rates can be achieved.

2.1 CFA piles features

CFA piles are a type of drilled foundation in which the pile is drilled to the final depth in one continuous process using a continuous flight auger. Equipment (such as schematically shown in Figure 2) is comprised of a mobile base carrier fitted with a hollow-stemmed flight auger which is rotated into the ground to the required depth of piling. The auger is fitted with a protective cap on the outlet at the base of the central tube and is rotated into the ground by the top mounted rotary hydraulic motor.

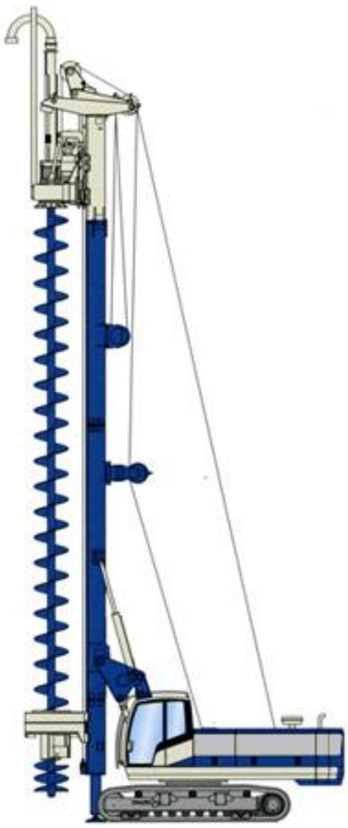


Figure 2. CFA Pile Rig

CFA piles are typically installed with diameters ranging from 0.3 to 0.9 m and lengths of even more than 30 m. European practice tends towards larger diameters (up to 1.5 m) (GeoSyntec Consultants, 2007).

The method is especially effective on soft ground and enables the installation of a variety of bored piles of various diameters that are able to penetrate a multitude of soil conditions. CFA piles are not normally viable in lower strength clays, unless a suitable end-bearing layer is available. Still, for successful operation of rotary auger the soil must be reasonably free of tree roots, cobbles, and boulders, and it must be self supporting.

CFA piles differ from conventional drilled shafts or bored piles, and exhibit both advantages and disadvantages over conventional drilled shafts. The main difference is that the use of casing or slurry to temporarily support the hole is avoided as the sides of the hole are supported at all times by the soil-filled auger. Drilling the hole in one continuous process is faster than drilling a shaft excavation, an operation that requires multiple lowering of the drilling bit to complete the excavation (GeoSyntec Consultants, 2007).

Because CFA piles are drilled and cast in place rather than being driven, as driven piles are, noise and vibration due to pile driving are minimized. Using CFA piles also eliminates amounts of splices and cutoffs. Soil heave due to driving can be eliminated when non-displacement CFA piles are used. A disadvantage of CFA piles when comparing to driven piles is that the available methods of the structural integrity and pile bearing capacity verification for CFA piles are less reliable than those for driven piles. Another disadvantage of CFA piles is that CFA piles generate soil spoils that require collection and disposal. Handling of spoils can be a significant issue when the soils are contaminated or if limited room is available on the site (GeoSyntec Consultants, 2007).

2.2 Construction sequence

Of course, this complex study of CFA piles foundation includes observations of construction methods. At the same time, this information is also useful for me in practical calculations as the result of drilled piles geotechnical bearing capacity calculations depends on the manner of their construction (this will be mentioned later in the 6th Chapter). So, present section provides an overview of the construction process of CFA piles.

2.2.1 Drilling

The key component of the CFA pile system contributing to the speed and economy of these piles is that the pile is drilled in one continuous operation using a continuous flight auger, thus reducing the time required to drill the hole. While advancing the auger to the required depth, it is essential that the auger flights be filled with soil so that the stability of the hole is maintained.

A variety of auger types may be used to drill the piles depending on the soil conditions encountered. Selecting the correct auger pitch is important, because for a given soil type an excessively large pitch could result in mining of the soil around the pile (GeoSyntec Consultants, 2007).

As the auger cuts the soil at the base of the tool, material is loaded onto the flights of the auger. The volume of soil through which the auger has penetrated will tend to take up a larger volume after cutting than the in-situ volume. Some of the extra volume appears also due to the volume of the auger itself, including the hollow center tube. Thus, it is necessary that some soil is conveyed up the auger during drilling. To maintain a stable hole at all times, it is necessary to move only enough soil to offset the auger volume and material bulking without exceeding this volume.

If the auger turns too rapidly, with respect to the rate of penetration into the ground, then the continuous auger acts as a sort of “Archimedes pump” and conveys soil to the surface. This action can result in a reduction of the horizontal stress necessary to maintain stability of the hole. Consequently, lateral movement of soil towards the hole and material loss due to overexcavation can result in ground subsidence at the surface, and reduced confinement of nearby installed piles.The bottom of Figure 3 illustrates an auger having too high rotation speed comparing with penetration rate; as a result, the auger feeds from the side with attendant decompression of the ground. The top of Figure 3 represents an auger having balanced auger rotation and penetration rates, so that the flights are filled from the digging edge at the base of the auger with no lateral "feed".

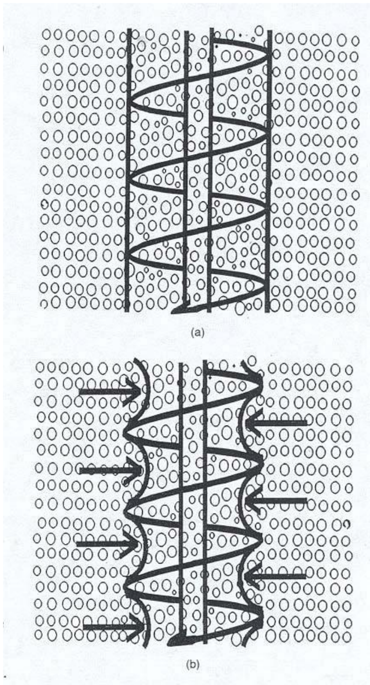


Figure 3. Effect of overexcavation using CFA piles

Controlling the rate of penetration helps to avoid lateral decompression of the ground inside the hole, loosening of the in-situ soil around the hole, and ground subsidence adjacent to the pile. The proper rate of penetration may be difficult to maintain if the rig does not have adequate torque and down-force to rotate the auger.

2.2.2 Grouting

When the drilling stage is complete and the auger has penetrated to the required depth, the grouting stage must begin immediately. Grout or concrete is pumped under pressure through a hose to the top of the rig and delivered to the base of the auger through the hollow center of the auger stem. Figure 4 shows the hole at the base of the auger stem.

Figure 4. Hole at the base of auger for concrete

The general grouting sequence is as follows:

- Upon achieving the design pile tip depth, the auger is lifted a short distance (typically 150 to 300 mm) and concrete is pumped under pressure to expel the plug at the base of the internal pipe and commence the flow. The auger is then screwed back down to the initial pile tip depth to establish a small head of concrete on the auger and to achieve a good bearing contact at the pile tip.

  It is essential that the grouting process begins immediately upon reaching the pile tip elevation; if there is any delay the auger may potentially become stuck and impossible to retrieve. To avoid this, sometimes a slow steady rotation of the auger is executed while waiting for delivery of grout; this rotation without penetration may lead to soil mining as described in the previous section and should be avoided. Another concern with excess rotation is degradation and subsequent reduction or loss of side friction capacity. So, the practice of maintaining rotation without penetration is not recommended. The best way to avoid such problems is to not start the drilling of a pile until an adequate amount of concrete is available at the jobsite to complete the pile.

- The concrete is pumped continuously under pressure (typically up to 2MPa measured at the top of the auger) while the auger is lifted smoothly in one continuous operation. This pressure replaces the soil-filled auger as the lateral support mechanism in the hole. When the grout pressure is applied, the grout also pushes up the auger flights and presses the soil against the auger.

  As the auger is slowly and steadily withdrawn, an adequate and controlled volume of concrete must be delivered at the same time to replace the volume of soil and auger being removed. If the auger is pulled too fast in relation to the ability of the pump to deliver volume, the soil will tend to collapse inward and form a neck in the pile.

  During grouting, the auger should be pulled with either no rotation or slow continuous rotation in the direction of drilling. A static pull with no rotation can help maintain a static condition at the base of the auger against which the grout pressure acts. Some contractors prefer to rotate the auger slowly during withdrawal to minimize the risk of having the auger flight getting stuck. Simultaneously, as the auger is lifted, the soil is removed from the flights at the ground surface so that soil cuttings are not lifted high into the air.

- After the concreting procedure is completed, any remaining soil cuttings are removed from the area at the top of the pile and the top of the pile is cleared of debris and contamination with a dipping tool.

- The reinforcement cage is lowered into the fluid concrete to the required depth immediately after the auger is removed. In general, reinforcement lengths between 10 and 15 m are considered feasible, depending on soil conditions. However, longer reinforcement has been used. A minimum cover of 75 mm is commonly adopted for most applications. Difficulties in placing the cage can arise if the concrete starts to set and loses workability. Soil conditions can also have an effect on the reinforcement placement, as a free draining sand or dry soil can tend to dewater the concrete rapidly and lead to increased difficulty in placing the cage.

A common practice in Europe is to utilize a small vibratory drive head to install the cage. The use of a vibratory drive head might lead to problems with cages that are not securely tied or welded, and might also produce segregation and bleeding if the concrete mix is not well proportioned.

3 MULTI-STOREY APARTMENT BUILDING "VITA NOVA"

The design and construction of any structure requires a thorough understanding of the properties of the type, load and performance criteria, location, and geometry of the proposed facility. The pile foundation preliminary analysis, design and calculation are also based upon the functional and structural significance of the building. So, the type, purpose, function and structures affect decisions regarding subsurface investigation programs and the type of foundation. Thereby, the idea of this chapter is to provide basic information about the main project decisions regarding the building, the design of which pile foundation was the focus of this work, and its structures. The main sources presented in this part of the work were project materials and documentation worked out by the JUVA Oy, adapted for Russian norms by JUVA RUS Ltd together with Genproject Ltd, and kept in the archive in the YIT Lentek JSC office.

3.1 General information about the building

The building being designed is a multi-storey residential house with built-in and attached office and public premises. The whole residential complex is called "Vita Nova". It consists of 7 sections of different heights which are shown in Figure 1.

As shown in Figure 5, the architectural structure of the whole complex has a symmetrical nature. In the plan the building is "V" shaped with distinctly marked corner. Sections N^{o}N^{o} 1, 2, 6, 7 (each has 23 floors), divergent along the streets, together with corner section N^{o} 4 (25 floors) compose the complex's altitude parts. Section N^{o} 3 and section N^{o} 5 have 2 floors. Sections N^{o}N^{o} 2, 6, 7 are identical and have a rectangular shape with dimensions in axes 33.2x17.9m.

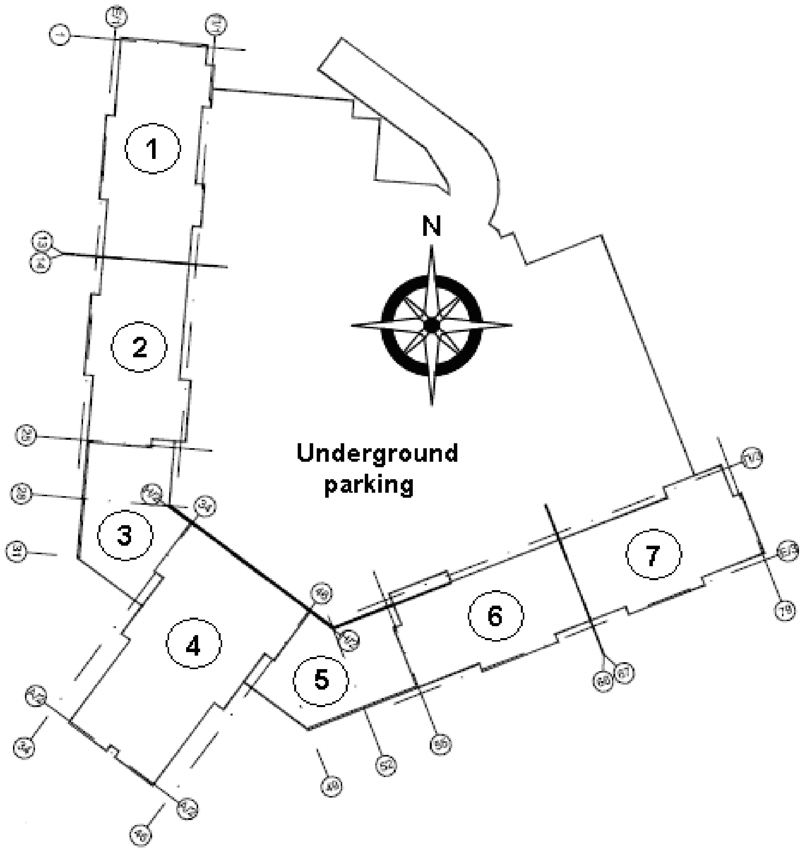


Figure 5. Residential complex "Vita Nova". Sections' location plan

3.2 Location of the building and brief site description

The area of design and construction is located in the north-western part of St.Petersburg. The address is: St.Petersburg, Primorsky district, Mebelnaya Street, parcel N^o 1 westerly from the crossroad with Turistskaya Street. The considered site has not previously been built up: there are neither permanent nor temporary old structures (buildings or supplying systems) on the building site. So, there is no need for any dismantling work before the start of construction.

The relief of the site is steady: absolute marks of ground surface are from +3.210 m to +6.440 m above the sea level. Such low absolute marks are typical for St.Petersburg which is located in the northern-western part of the Nevsky lowland terrain.

The entrance to the territory of the residential complex is accessed from the sides of Mebelnaya and Kamyshovaya streets. The master plan with all the necessary designations is presented in Appendix 1. The total area of the described site is 19 834 m^2, with a building area is 4 784 m^2. Thus, the building density rate is, as follows, 24%.

3.3 Architecture

Architectural and planning solutions, due to absence of strict environmental factors and conditions, are conditioned by economical and technological features of existing buildings and design tasks and requirements (Genprojekt Ltd, 2007).

3.3.1 Facades and exterior finishing

Pictures of perspective views, worked out by YIT Lentek's designers, which show the whole design of the building, are presented in Appendix 2. All the peculiarities of the building's appearance can be seen there.

Facades have vertical and horizontal divisions, emphasizing the functional nature of the building, and glazed balconies and loggias. Decorative white and grey plaster is used for facade finishes. For the 1st floor, exterior finishing in grey coloured (natural) concrete stone is used.

3.3.2 Functional and planning solutions

The building has a basement floor which is partly occupied with household pantries for residents. Pump units, individual heating units, cable spaces and metering units are also situated on the base floor.

Underground parking (total area 5 957 m²) which provides 249 places for cars is attached to the building. The entrance to the underground parking is accessed by a double-entry ramp which is separated from entrances to dwellings and offices in accordance with Russian Sanitary Standards.

The built-in office and public premises (café, club rooms and technical areas) are located on the 1st floor in all sections and on the 2nd floor in sections N^{o}N^{o} 3, 4 and 5. Their total area is 4 784 m². Each built-in facility has its own private entrance to ensure compliance with fire safety requirements.

It is important to note, that commercial premises located on the first floors of residential buildings, are examples of rare cases when the interests of all parties (business structures – potential tenants, developers and residents) coincide. It is difficult for developers to sell apartments on the first floors, because not many people want to live there, but at the same time original planning and positioning of these areas as commercial is interesting enough for them. First floor apartments have always had lower demand from buyers and there are many reasons for this: the first floors are the least protected from external intrusion, street noise, and other "minor" troubles. As usual, there is also no balcony in an apartment located on the first floor. People who live higher like to have small shops, a pharmacy, a hairdressing salon or similar convenience downstairs.

Dwelling spaces are on the 2nd floor and higher and in corner section N° 4 on the 3rd floor and higher. Apartments have from 1 to 4 rooms of different areas. The total amount of apartments projected inside the building is 853. Their total area is 53 634.06 m².

3.4 Structures

The foundation slab is monolithic, with a constant cross section's thickness of 800 mm. The slab rests directly on pile foundation. More details about foundations will be given later.

Taking into consideration the high groundwater level (section 3.3 of this thesis) and to increase reliability of constructive solutions, the basement is implemented monolithically. Cast-in-place external and internal walls and columns are rigidly jointed with the ceiling slab, thus forming a system of load-bearing frames and load-bearing walls in both transversal and longitudinal directions.

The constructive scheme of the superstructure is a composite (precast-cast-in-place) system with load-bearing walls arranged mainly in the transversal direction, but also in the longitudinal one. Load-bearing elements of floors and roofing are cast-in-situ reinforced concrete slabs with a 220 mm thickness.

Vertical load-bearing elements in the basement and on the 1st floor are cast-in-situ columns with a cross-section of 600x400 mm, and precast walls with a 160 mm thickness with cast-in-situ joints. Cast-in-situ external and internal walls (160 mm) are load-bearing elements on the 2nd floor; the joints are also cast-in-situ (rigid). From the 3rd floor and higher, external and internal load-bearing walls are precast reinforced concrete panels with a 160 mm thickness (JUVA RUS Ltd., 2007).

4 THEORETICAL BASICS OF CFA PILES FOUNDATION DESIGN AND CALCULATION

As it was mentioned earlier, one purpose of this thesis is to study and describe the process of pile foundation design of the residential complex and calculating CFA piles bearing capacity. This chapter provides the results of the theoretical study: a general overview of the theoretical basis of this pile type design and calculation.

The purpose of a pile foundation is to transmit the loads of a superstructure to the underlying soil while preventing excessive structural deformations. Design of a pile foundation involves solving the complex problem of transferring loads from the structure through the piles to the underlying soil. The capacity of the pile foundation is dependent on the material and geometry of each individual pile, the pile spacing (pile group effect), the strength and type of the surrounding soil, the method of pile installation, and the direction of applied loading (axial tension or compression, lateral shear and moment, or combinations). In general, the ultimate capacity is limited by either the structural strength of the pile shaft or the capacity of the supporting soil.

4.1 Limit states for design

The design method presented is based on an Allowable Stress Design (ASD) for geotechnical conditions, in which a factor of safety is applied to ultimate limit state conditions to obtain allowable resistance values for design. The structural design of the pile is in accordance with those for other reinforced concrete structural elements. In general, there are three limit state conditions that must be satisfied for design of CFA and other deep foundations:

<u>1. Geotechnical Ultimate Limit State (GULS)</u>

The pile should have a load resistance that is greater than the expected loads (service loads) by an adequate margin to provide the required level of safety

(safety factor). For axial compressive loads, the GULS is shown in Figure 6. For lateral loads, the GULS may be defined as a push-over failure of the foundation or alternately as some deflection limit at which collapse of the structure above the foundation may occur. Uplift loading conditions and group behavior for axial, rotation, or lateral are additional geotechnical ultimate limit states that may control some cases.

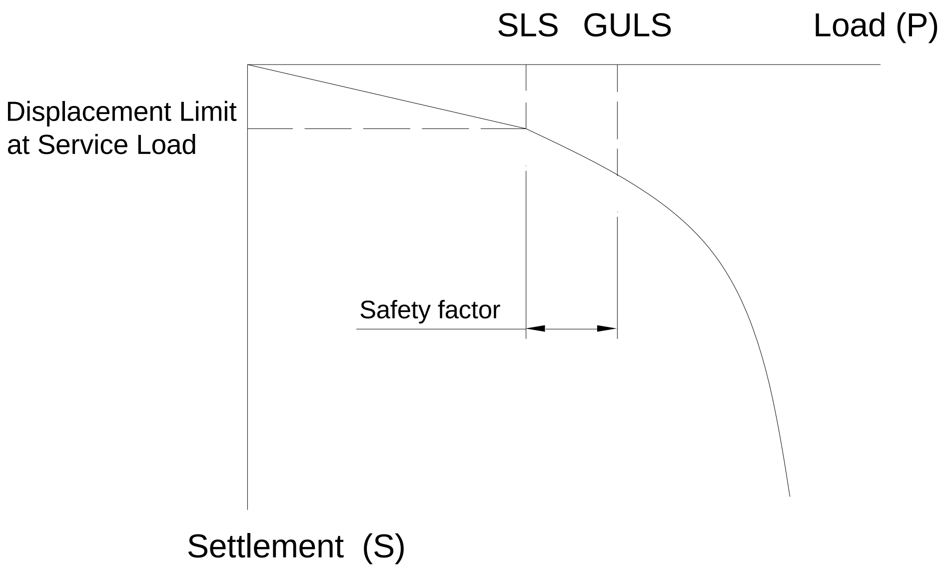


Figure 6. GULS and SLS for axial load on single CFA pile

It is noted that the GULS is often referenced using the words “capacity” or “failure”, which are a poor choice of words, because no collapse or condition of plunging may exist at the GULS, and the pile may have a capacity to support additional loads beyond the GULS. The state of deformation associated with the GULS is not to be confused with deformations at service load levels. The GULS provides a definition of foundation resistance.

For the design of piles using ASD, the resistance computed at the GULS is compared with service loads. To demonstrate that the foundation will support the design load with adequate safety against bearing resistance failure, the following inequality (1) shall be satisfied for all ultimate limit state load cases and load combinations:

$$N < \frac{F_d}{SF} \tag{1}$$

where: N = value of total design load;

F_d = resistance at GULS;

SF = factor of safety.

If inequality (1) is satisfied for the most loaded piles, it will be likely satisfied for other ones. Theoretical formulae for total design load for each individual vertical pile, given in SNiP 2.02.03-85, is presented bellow (equation 2).

$$N = \frac{N_d}{n} \pm \frac{M_x y}{\sum y_i^2} \pm \frac{M_y x}{\sum x_i^2} \tag{2}$$

where: N_d = design value of compressive force, kN;

M_x, M_y = design value of bending moments respectively to the principal central x and y axes of piles layout plan, kNm;

n = total amount of piles;

x_i, y_i = distance from the principal axes to the axis of each pile of the foundation, m;

x, y = distance from the principal axes to the axis of each pile, for which design load is calculated, m.

Traditionally, F_d, the GULS pile bearing capacity, may be estimated from calculations based on analysis of soil mechanics, static sounding of the ground and verified with static or dynamic load tests. Now some words about static and dynamic load tests because other options will be considered later: more information about static sounding of the ground is presented in Chapter 5 and Chapter 6 provides my practical calculations of the CFA pile bearing capacity. The pile load tests are intended to validate the computed capacity for a pile foundation and also to provide information for the improvement of design rationality. This reduction in uncertainty may allow the use of lower design factors of safety employed in the ASD methodology. The test results can also be used to evaluate correlations of side and base resistance with site-specific soil parameters, and to revise or improve the design.

Static testing involves the application of static loads and the direct measurement of piles movements. It is the most fundamental form of pile load test. Loading is applied to the test pile using a calibrated hydraulic jack, and where required, a calibrated load cell measures the load. Direct measurements of pile displacement under the applied loading are taken by reading deflectometers. Test results are presented unconventional graphical format showing the applied load versus pile head displacement. At some load, the movement of a pile becomes unacceptable (the unacceptable value for multi-story apartment building is 20% of limit value of absolute settlement) and this load is considered as a pile bearing capacity.

Dynamic load testing is a quick method to evaluate the bearing capacity of a pile for loads similar to the design ones. It can be used for both prefabricated and cast-in-place piles. It is considerably faster than the static load test. Dynamic load testing is carried out with two identical bolt-on strain and acceleration sensors attached to a section of pile with anchor bolts. The pile is then struck with a driving hammer or a separate drop weight. A hammer mass of about 1 to 2% of the test load is generally sufficient. The generated compressive stress wave travels down the piles and reflects from the pile toe upward. The stress waves, which are picked up by the sensors, are processed and automatically stored in the computer for further analysis and reporting. Pile and soil data are modeled and a response is calculated.

2. Service Limit State (SLS)

The pile should undergo deformations at service load levels that are within the tolerable limits appropriate for the structure. The actual definition of the service limits should be determined by a rational assessment of the sensitivity of the structure to deformations. Short-term deformations for transient loadings are a function of the mobilization of pile resistance. However, long-term settlements under structural dead loads are a function of group settlements and should be computed accordingly.

3. Structural Ultimate Limit State (SULS)

After the design has been selected to satisfy considerations of geotechnical strength (GULS) and serviceability (SLS), the final structural design of the piles and pile cap must be completed. This step involves the final lateral pile analysis or verification and the pile structural design, including reinforcement and grout or concrete material requirements. The pile must have sufficient structural capacity when the pile is subjected to combined axial and flexural loads, so that structural yielding of the pile is avoided.

4.2 Design procedures

The recommended procedure for performing a foundation design of CFA piles is outlined below. This outline was developed by me after having studied the process of the design of pile foundation for the “Vita Nova” apartment building and may be considered as suitable for routine foundation design projects for structures like this.

1. Development of idealized profile

Using the borings and geological descriptions from the site investigation program, the site is conventionally divided into zones for foundation design according to similarities in the soil profile and properties, and idealized geotechnical design profiles for each representative zone at the site are established. The differing of zones should adequately cover the range of conditions at the site, and the designers may develop multiple profiles for each zone to evaluate the possible range of geotechnical (and groundwater) conditions within each identified zone.

2. Development of geotechnical design parameters

For each stratum defined in the idealized profiles, the geotechnical design parameters for each layer are evaluated. This includes:

a. Soil strength parameter values (such as shear strength, friction angle and cone tip resistance). These strength parameters will be used either directly or through correlations to estimate unit side-shear and end-base resistances.
b. Soil stiffness or modulus and other parameters related to deformation characteristics.
c. Other soil properties that may be needed for design, such as unit weights and index tests for classification.

3. Obtaining the loadings for the foundation

The design loadings will mostly include several cases of both axial and lateral loads. Many different load combinations exist of dead loads, temporary loads, wind and snow loads, etc. Some combinations will include extreme event loadings. Analyses of the load combinations will reveal the maximum axial and lateral loads imposed to the pile and represent the critical design cases.

4. Safety factors for design

The safety factors are suggested for general use in design for strength, and differ depending on the level of site specific testing and quality control. Large values may be applied in cases where unusual variability in subsurface conditions exist, difficult construction conditions are expected, or if other considerations for the structure require this. For example, where base resistance contributes a large portion of the axial resistance and the properties of the bearing stratum are quite variable, it may be appropriate to use a significantly greater safety factor on base resistance than side-shear even where a load test is performed.

5. Trial design of individual CFA pile

The number of piles used to support the foundation loadings will be determined by individual pile bearing capacity. At this point in the process, experience and preliminary calculations should suggest some reasonable values of nominal axial and lateral resistance for a single pile so that an efficient layout can be developed later. A range of diameters and lengths may be considered for groups of piles, as it is feasible to consider foundations with larger numbers of smaller capacity piles vs. fewer numbers of larger capacity piles.

A. Design for Lateral Loading

Lateral analyses may only be needed if lateral loads are significant. As a guide, lateral shear forces on vertical piles that are less than about 9 to 22 kN per pile for piles with diameter up to 915 mm would probably not justify lateral analysis at this point in the design process, and the designer could skip the lateral analyses and move on to design for axial load (GeoSyntec Consultants, 2007).

Select a diameter that is sufficient to provide the necessary nominal lateral load resistance and service load requirements for deflection. In general, smaller diameter and greater length piles tend to be more economical than larger diameter, shorter length piles with similar axial resistance. So, pile diameter is often controlled by lateral shear and bending moment considerations. Lateral load considerations almost never control the length of CFA.

When lateral loads are significant, the final design of CFA piles for lateral loading is typically controlled by structural design considerations and the necessary flexural strength and reinforcement. Very high bending stresses combined with unsupported length may preclude the use of CFA piles at all.

The lateral load analysis of a trial pile design should proceed as follows:

1. Select a trial pile diameter and length. Select a trial longitudinal reinforcement. A longitudinal reinforcement with a cross-sectional area of around 1% of the pile cross sectional area is typically a good initial value to consider. Construct a computer model for the load conditions likely to be most critical for lateral load considerations.

2. With the pile modeled as a linear elastic beam, foundation strength conditions are evaluated by computing the foundation response of the pile due to service loads multiplied by a safety factor to ensure that the pile embedment into the soil has adequate reserve capacity. Service load deflections or structural

strength requirements generally control lateral load design, but this “push-over” type analysis is performed to ensure that adequate reserve strength exists with respect to the soil resistance. It also should be noted that there may be several different load combinations to be evaluated, although there is usually a clearly dominant lateral loading case.

3. Verification that the amount and depth of longitudinal reinforcement is adequate for the maximum bending stress computed with the computer program as in step 2.

4. With the pile modeled as a nonlinear reinforced concrete beam, foundation deflections due to service loads are evaluated. If the deflections are larger than the service load requirements, the pile diameter may need to be increased. If the lateral loads are very high, the use of structural steel pipe or H sections for reinforcement in CFA piles, or selection of alternative deep foundation systems should be considered.

B. Design for Axial Loading

After the diameter is selected, determine the length of pile required to provide the necessary axial resistance. The allowable axial resistance (ultimate axial resistance divided by the factor of safety) is compared to the service loads to ensure that the design meets strength requirements.

The most efficient approach for design is to check the first ultimate limit state conditions and then serviceability. These limit states for design were fully described in the previous section.

6. Pile layout

The simplest pile layout is one without batter piles. Such a layout should be used if the magnitude of lateral forces is small. In this case, the designer begins by dividing the largest vertical load on the structure by the reduced computed pile capacity (this provides a good starting point to determine an initial layout) to obtain the approximate number of piles.

A uniform pile grid should be developed based on the estimated number of piles, the required pile spacing and the area of the pile cap. An ideal layout for flexible structures will match the pile distribution to the distribution of applied loads. This match will result in equal loads on all piles and will minimize the internal forces in the structure because the applied loads will be resisted by piles at the point of loading.

After the preliminary layout has been developed, the remaining load cases should be investigated and the pile layout revised to provide an efficient layout. The goal should be to produce a pile layout in which most piles are loaded as near to capacity as practical for the critical loading cases with tips located at the same elevation for the various pile groups within a given monolith. Adjustments to the initial layout by the addition, deletion, or relocation of piles within the layout grid system may be required. Large concentrations of piles under the centre of the pile cap should be avoided. This could lead to load concentration resulting in local settlement and failure in the pile cap (Dalmatov, 2002).

7. Pile group capacity

The geometry and layout of the pile group will affect individual loads per pile. For pile groups with complex 3-D modeling load conditions or soil layering, the pile group capacity will require analyses using computer-based pile group analysis methods. Computer-based tools can be effectively used to optimize foundation layout and load distribution to the piles. To illustrate the computing of pile group capacity, in Appendix 3 I have placed pictures showing the results of computing of internal forces which occur in drilled piles under the action of one of the load combinations obtained by the engineers with the help of SCAD Office 11.1.

8. Pile group settlements

Settlements of pile groups should be calculated to give the designer a perception of how the structure will perform and to check that these calculated settlements are within acceptable limits. These limits are set in accordance with SNiP 2.02.01-83* “Foundations of Buildings and Structures” and for multistorey buildings with reinforced bearing walls the limit value of absolute settlement is 10 cm.

Group settlement may be evaluated by hand calculations for simple load conditions and soil layering. In this case, SNiP 2.02.03-85 “Pile Foundations” dictates that calculation of settlement of pile foundation usually should be done as for “conventional” foundation on a natural basement. The boundaries of the “conditional” basement (see Figure 7) are defined as follows:

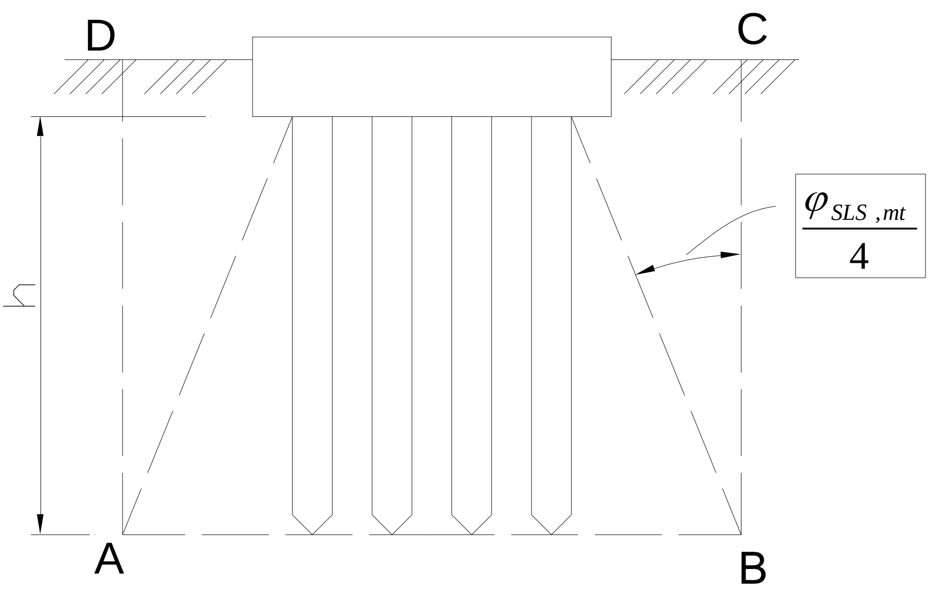


Figure 7. Determination of the “conditioned” basement boundaries for pile foundation settlement calculation

- bottom – AB surface, passing through the lower ends of the piles;
- on both sides - the vertical surfaces AD and BC distanced from the external faces of extreme vertical rows of piles at a distance of $h \cdot tg(\varphi_{SLS,mt}/4)$. Here $\varphi_{SLS,mt}$ is the average design value of the internal friction angle of the soil, determined by my formula:

$$\varphi_{SLS,mt} = \frac{\sum_{0}^{h} \varphi_{SLS,i} h_i}{\sum h_i} \qquad (3)$$

- top - the ground surface CD

For pile groups with complex 3-D load conditions or soil layering, the pile group capacity may also require computer-based pile group analysis methods. However, most settlement analysis methods are based on empirical methods and give a rough approximation of the actual settlement.

9. Pile group lateral behavior

This may be sufficient for most pile groups, and a sensitivity analysis may be considered to determine primarily the effect of the pile-head fixation. However, for pile groups with complex soil-pile interaction or cap designs, the pile group lateral capacity may also require computer-based pile group analysis methods.

10. Structural design of piles and pile cap

Normally, pile foundation of multistorey apartment building consists of pile cap and a group of piles. The pile cap distributes the applied load to the individual piles which, in turn, transfers the load to the bearing ground. The individual piles are spaced and connected to the pile cap or tie beams and trimmed in order to connect the pile to the structure at cut-off level.

After the design has been selected to satisfy considerations of geotechnical strength (GULS) and serviceability (SLS), the final structural design of the piles and pile cap must be completed. This step will involve the final lateral pile analysis or verification and the pile structural design, including reinforcement and grout or concrete material requirements.

If the pile group is analyzed with a flexible base, then the forces required to design the base are obtained directly from the structure model. If the pile group is analyzed with a rigid base, as it was done in initial design of the “Vita Nova” residential complex driven piles foundation (Figure 8 shows the rigid connection of piles and pile cap provided by 700 mm of exposed reinforcing rebars anchored in the pile cap), then a separate analysis is needed to determine the stresses in the pile cap. The example of this kind of computing in SCAD Office

11.1 executed by engineers for the pile cap of “Vita Nova” residential complex is presented in Appendix 4.

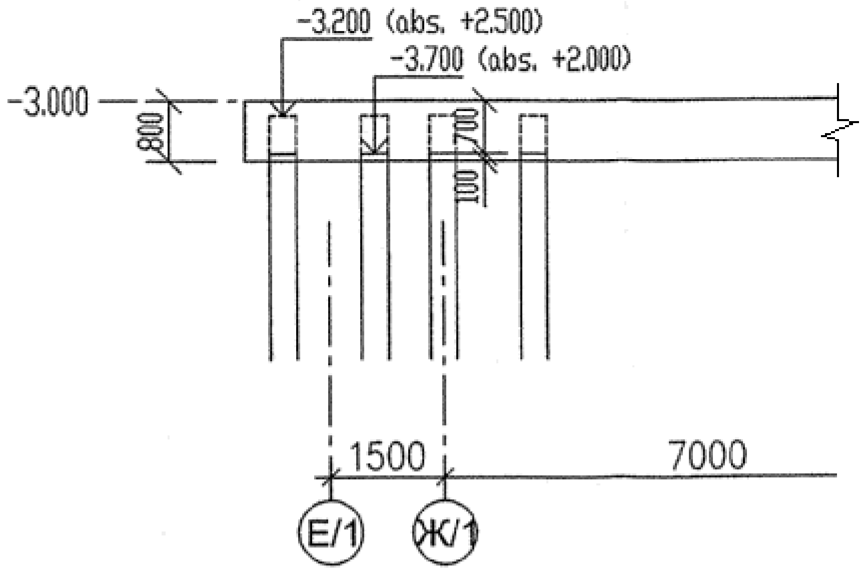


Figure 8. Scheme of rigid connection of piles and pile cap accepted in project of “Vita Nova” residential complex

The applied loads and the pile reactions should be in equilibrium. Appropriate fictitious supports may be required to provide numerical stability of some computer models. The reactions at these fictitious supports should be negligible.

5 GEOLOGICAL CONDITIONS OF THE SITE

Geotechnical investigations are performed to evaluate those geologic and soils conditions that affect the safety, cost effectiveness, design, and execution of a proposed engineering project. Insufficient geotechnical investigations, faulty interpretation of results, or failure to portray results in a clearly understandable manner may contribute to inappropriate design, delays in construction schedules, modifications in construction costs, use of unsuitable material, environmental damage to the site, postconstruction remedial work, and even failure of a structure.

5.1 Geological investigations programme

Whereas the properties of man-made materials (e.g., brick, concrete, steel) can be varied on demand, soil and rock formations have already been provided by Mother Nature, and in many cases, have been in place for many thousands of years. Thus, the properties of soil and rock must be evaluated through a program of limited testing and sampling.

A detailed investigation was performed by LenTISIZ JSC for the purpose of detailed site characterization to be used for design. As is frequently done, the design phase investigation was performed in two stages: in March and August 2007.

The initial stage investigation is typically performed early in the design process prior to defining the proposed structure elements or the specific locations of foundations. Accordingly, this stage of investigation typically includes a limited number of borings and testing sufficient for defining the general stratigraphy, soil characteristics, groundwater conditions, and other existing features which are important for foundation design. The following types of work were executed in the 1st stage of the survey (in March 2007):

- Drilling of 8 boreholes (brh. 1–8) of depths from 35.00 m to 40.00 m, the total drilling meterage – 305.00 m;
- Static sounding of the ground in 7 points (ssp. 1–7) to a depth from 26.00 m to 31.20 m, the total sounding meterage – 181.7 m;
- Sampling for laboratory tests: 57 disturbed samples and 72 undisturbed samples for ground properties testing and 3 groundwater samples for water chemical composition testing (LenTISIZ JSC 2007).

Subsequently, after the location of structure foundations and other design elements had been determined, the second investigation phase was performed to obtain site specific subsurface information at the final substructure locations for design purposes and to reduce the risk of unanticipated ground conditions during construction. The following types of work were executed during the 2nd stage of the survey (in August 2007):

- Drilling of 13 boreholes (brh. 9–21) of depths from 40.00 m to 48.00 m, the total drilling meterage – 558.00 m;
- Static sounding of the ground in 13 points (ssp. 9–21) to a depth from 28.00 m to 31.40 m, the total sounding meterage – 392.8 m;
- Sampling for laboratory tests: 84 disturbed samples and 164 undisturbed samples for ground properties testing and 2 groundwater samples for water chemical composition testing (LenTISIZ JSC 2007).

The total number and depths of borings provide adequate coverage and are sufficient to reasonably define the subsurfaces in the site area. Archive materials of borings of recent years' geological surveys of this site and adjacent territories were also studied and analysed during the investigations (brh. N°N° 213, 280, 282, 283 and 285) as the review of existing data is actually the first step in the investigation process. A plan of the locations of all drilled boreholes and static sounding points is presented in Appendix 5.

The purpose of laboratory tests is to investigate the physical and hydrological properties of soil, determine index values for identification and correlation by means of classification tests, and define the engineering properties in parameters usable for the design of foundations. Soil samples obtained for laboratory tests and analysis were of two main categories: disturbed and undisturbed. Disturbed samples are those obtained using equipment that destroy the macro structure of the soil but do not alter its mineralogical composition. Undisturbed samples were obtained with minimization the disturbance to the in-situ structure and moisture content of the soils. It should be noted that the term "undisturbed" soil sample refers to the relative degree of disturbance to the soil's in-situ properties (Mayne, Christopher, DeJong, 2001).

Having obtained the data from the field investigation and laboratory testing program, the focus turned to the reduction and evaluation of these data, the definition of subsurface stratification, groundwater conditions and appropriate soil design parameters, and the presentation of the findings in a geotechnical report. This report with the survey results made by LenTISIZ JSC was accepted and approved for further application in the residential complex designing by Geological and the Geodesic Department of the City Planning and Architecture Committee. Thus, the geotechnical investigations provide all the data concerning the ground and the groundwater conditions at the construction site necessary for a proper description of the essential ground properties and a reliable assessment of the characteristic values of the ground parameters to be used in design calculations.

5.2 Geological structure of the site

To cover and systemize the results of the geological investigations of the site which are presented in the previously mentioned report by LenTISIZ JSC, I have composed a table called “Geological structure of the site” (Appendix 6). It shows the ultimate and average values of thickness of all detected under the 1st section’s construction spot layers and includes information about their geological age and genesis and their description.

According to boring and static sounding data, the geological structure of the site involves 12 different geological soil elements to a depth of 48.00 m. The elements were distinguished on the basis of their structural composition and physical and mechanical properties. Numbers of the elements were accepted in accordance with archive materials of LenTISIZ JSC.

Geological sections VI–VI between brh.9 and brh.10 and VII – VII between brh.8 and brh.6 show the nature of bedding and thickness of particular lithological elements within the 1st section’s construction spot (Appendix 7). To obtain the idealized geotechnical profile of the site, which will be used for calculation of CFA pile bearing capacity, these geological sections should be considered together with average values of thickness of each soil element, presented in table in Appendix 6.

Basic physical and mechanical properties of the soils, which are taken into consideration in foundation designing calculations, are presented in Appendix 8.

5.3 Static sounding of the ground

Static sounding of soils is one of the most effective methods of soils investigation in their natural occurrence. Application of static sounding allows the study of the composition, condition and properties of complex, unpredictable soils of St. Petersburg in detail, and this usually turns into a significant savings in material, energy, time and money.

Static sounding of the ground was carried out by Geozond Ltd with heavy type testing machine with a total weight about 18 tonns. Software and measuring converters (cones, loggers) produced by Fugro Engineers b.v. were used for sounding purposes.

Field static sounding tests of the ground were carried out in conjunction with other methods of engineering–geological research for:

- more accurate allocation of engineering-geological elements (thickness, upper and bottom boundaries of soils of different composition and condition);
- determination of the soil homogeneity in both horizontal and vertical directions;
- approximate determination of property characteristics of the soils; determination of soil resistance under the pile end and along its lateral surface;
- evaluation of possible depth of pile sinking.

Static sounding of the ground is executed by pressing the probe into the soil, with simultaneous continuous measuring of soil resistance values under the tip of the cone and on the lateral surface of the probe. During field static sounding tests of the ground, specific ground resistance under the cone of the probe (cone resistance) and specific ground resistance on the lateral surface of the probe (sleeve friction) are determined.

5.4 Groundwater situation

Ground water studies of any building site include observations and measurements of flows from springs and of water levels in existing production wells, boreholes and piezometers. This information is used with site and regional geologic information to determine water table or piezometric surface elevations and profiles, fluctuations in water table elevations, the possible existence and location of perched water tables, depths to water-bearing horizons, and direction and rate of seepage flow. Complex investigations are made only after a thorough analysis is made of existing or easily acquired data. Results from ground water studies provide data needed to design dewatering and seepage control systems at construction projects, indicate the potential for pollution and contamination of existing ground water resources due to project operation, show potential for interference to aquifers by the construction of a project, and determine the chemical and biological quality of ground water and that relationship to project requirements.

Hydrogeological conditions of the “Vita Nova” residential complex site are described in this work on the basis of information given in the geological report by LenTISIZ JSC. The groundwater situation to the investigated depth is characterized by the presence of an aquifer which is confined to the Quaternary sediments. During the investigations, in March and August 2007, non-pressure groundwater was revealed in filled, silt and peaty soils as well as in silty sands (elements №№ 3 and 4a) and in sandy interlayers of elements №№ 4, 5, 6, 7 and 10.

Average annual groundwater level is at the depth of 1.10–2.50 m. In August 2007, in boreholes №20 and №21 groundwater was also revealed in anthropogenic soils at the depth of 0.30–0.70 m. Taking into consideration the previous years' measurement results (July 1990 and April 1993), this level can be set down as a maximal annual level. During intensive rainfall and snowmelt periods, groundwater can be expected close to the surface, and open-water may appear in the depressions. Pressurized groundwater was not discovered while investigating the site.

Chemical tests of the groundwater have shown that it is slightly aggressive to concrete W4 by pH-value and also medium aggressive to concrete W4 and slightly aggressive to concrete W6 by the content of carbonate.

6 CALCULATION OF THE BEARING CAPACITY OF A SINGLE CFA PILE

Calculation of the GULS bearing capacity of a single CFA pile on the basis of Russian Building Norms and Rules is one of the declared aims of this work. In general, the final result of this calculation can not be considered as an exact value of the pile's bearing capacity unless field load testing is done. So, this is done to get the opportunity for approximate evaluation of the bearing capacity.

The pile is considered on the idealized geological profile of the site developed earlier (this is described in Section 5.2). As was mentioned earlier, thicknesses of the soil layers are accepted as average values presented in the table found in Appendix 6. The calculation will be made for a pile which has a diameter 0.3 m and the same length as initially projected driven piles have - 28 m. The absolute mark of the pile connection with pile cap is also taken as for drilled piles: +1.900 (see Figure 8). Seating of the pile on idealized geological profile is shown on scheme in Appendix 9.

Bearing capacity F_d (kN) of CFA piles, working in compressive load, should be calculated by the given in SNiP 2.02.03-85 formulae:

$$F_d = \gamma_c(\gamma_{cR}RA + u\sum\gamma_{cf}\, f_i h_i)\,, \qquad (4)$$

where: γ_c = coefficient of a pile working conditions;

γ_{cR} = coefficient of the working conditions of the soil under the lower end of a pile;

R = design value of soil resistance under the lower end of a pile, kPa;

A = cross section area of a pile, m²;

u = cross section perimeter of a pile, m;

γ_{cf} = coefficient of soil conditions on the lateral surface of a pile, depending on a pile-shaft formation method and conditions of concreting;

f_i = design value of soil resistance of each soil layer on the lateral surface of a pile, kPa;

h_i = thickness of each soil layer which is in contact with the lateral surface of a pile, m.

Analysis of the given formulae allows us to say that the calculated bearing capacity of a pile is the sum of two terms, where the first one is the base or end bearing component, and the second one is the frictional shaft component. This is schematically shown in Figure 9.

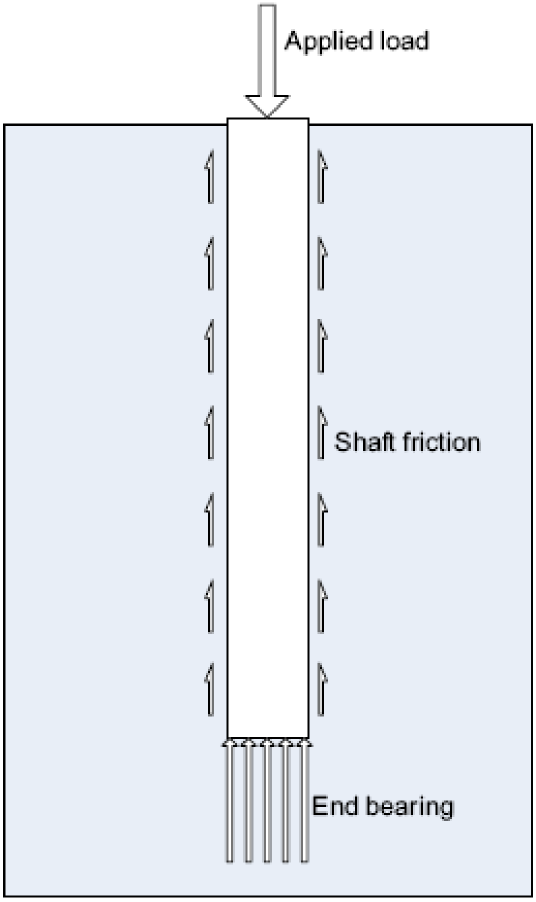


Figure 9. CFA pile bearing capacity components

For convenience, I will divide the calculation in 3 stages:

1) end bearing component calculation;
2) shaft friction component calculation;
3) final result obtaining.

1. End bearing component calculation

Firstly, I should determine the coefficient of the working conditions of the soil under the lower end of the pile - γ_{cR}. In accordance with SNiP 2.02.03-85 this value is 1.0 for all cast-in-place piles, except the piles with camouflet widenings and piles with widenings being concreted under the water.

The second step is to determine the design value of soil resistance under the lower end of the pile (*R*). This value is accepted depending on the type of the base soil. As shown in the drawing in Appendix 9, the lower end of the trial pile is deepend into the hard silty clay (geological soil element N^{o} 11). In this case, the design value of soil resistance under the lower end of the pile should be taken from Table 7 of SNiP 2.02.03-85 subject to the depth of the lower end of the pile and the liquidity index of the base soil. As the relative mark of the pile's end is –28.000 and the value of hard silty clay liquidity index is –0.42 (see Appendix 8), soil resistance under the lower end of the pile is 4200 kPa.

As the diameter of a pile is 0.3 m, then the cross section area is 0.071 m². So, the result of this phase of calculation is:

$$\gamma_{cR} \cdot R \cdot A = 1.0 \cdot 4200kPa \cdot 0.071m^2 = 298{,}2kN$$

2. Shaft friction component calculation

This method of calculation of the shaft friction component is based on partitioning the ground stratas, which are cut through by the pile, into homogeneous layers of thickness (h_i) not more than 2 meters. For example, the thickness of silty loam (geological soil element №4) on the idealized geological profile is 6.80 m, so, it is divided into 4 layers with thicknesses of 2.0 m, 2.0 m, 1.8 m and 1.0 m. Then, taking the absolute mark +1.900 as a relative mark 0.000, I determine the average depth of each layer (the depth of its middle). And finally, the values of soil resistance of each soil layer (f_i) are obtained from Table 2 of SNiP 2.02.03-85 subject to the average layer's depth, the type and the liquidity index of soil.

The coefficient of soil conditions on the lateral surface of a pile (γ_{cf}) for each particular soil type is determined in accordance with Table 5 of SNiP 2.02.03-85, depending on the pile-shaft formation method. In view of a very high degree of water content in some geological soil elements, the execution of a CFA pile on the considered site is supposed to be done using a casing pipe. In this case, γ_{cf}–value is 0.6 for clay and 0.7 for all other soil types.

The whole calculation is presented in Appendix 10. Multiplying the result of this calculation by the value of the cross section perimeter of a pile (0.94 m), I get the result of the 2nd stage of CFA pile bearing capacity calculation:

$$u\sum\gamma_{cf} f_i h_i = 0.94m \cdot 536.18kPa \cdot m = 504kN$$

3. Obtaining the final result

For getting the final result of the calculation coefficient of a pile working conditions (γ_c) should be determined. This coefficient depends on the type of base soil and its degree of water saturation, in this case γ_c=1.0. Finally, bearing capacity F_d (kN) of a single CFA pile is calculated:

$$F_d = 1.0 \cdot (298.2 + 504) = 802.2kN$$

Thus, the result of the calculation of bearing capacity of a single CFA pile, made in accordance with SNiP 2.02.03-85 “Pile Foundations”, is 802.2kN, or approximately 80 tonns. The whole calculation is done without using any computer-based programmes and is mainly based on analysis of soil mechanics. The result should not be considered as an exact value of the pile’s bearing capacity and needs to be verified by computer-based calculations and, mainly, by field load testing. Thus, this value can serve as a starting point in determining bearing capacity of a single CFA pile of a given diameter and length.

7 SUMMARY

At the beginning of writing this thesis, the mechanism and main features of continuous flight auger piling, which has high level of productivity and is characterized with a high quality of shaft concreting because of supplying it under pressure, were studied. Then, using the project materials and documentation of the “Vita Nova” residential complex, provided by the YIT

Lentek JSC, other research works and theoretical manuals, I have described and systemized the process of pile foundation design for a multi-storey apartment building starting from the analysis of geological conditions and finishing with structural design of the pile cap. The importance and targets of each design stage were defined and described and also some graphic materials are attached for illustration. These guidelines for pile foundation design for a typical multi-storey apartment building, written in English, are useful not only for Russian building or design companies, which often cooperate with foreign ones, but can also be applied for further research.

This thesis also contains the elements of my engineering practice which includes analysis of geological conditions of the site, development of idealized geological profile and hand calculation of the bearing capacity of a single CFA pile on the basis of Russian Building Code. This type of calculation was carried out by me for the first time and it was not easy because of my lack of engineering experience.

The structure/pile/soil system is highly indeterminate and nonlinear. Historically, design methods have been based on numerous simplifying assumptions that make the analytical efforts tractable for hand calculations. So, this hand calculation, just like the majority of hand calculations nowadays, likely does not show the exact value of a single CFA pile bearing capacity. The obtained result should be defined more exactly by computing, as modern computer-based calculations, in which many of the simplifications of the classical design methods are no longer necessary, allow accurate enough results to be obtained. Thus, the calculation was done to get the opportunity for approximate evaluation of a single CFA pile bearing capacity and serves as starting point in determination of bearing capacity of a single CFA pile of a given diameter and length for further considerations and comparisons with other variants of piling.

LIST OF FIGURES

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