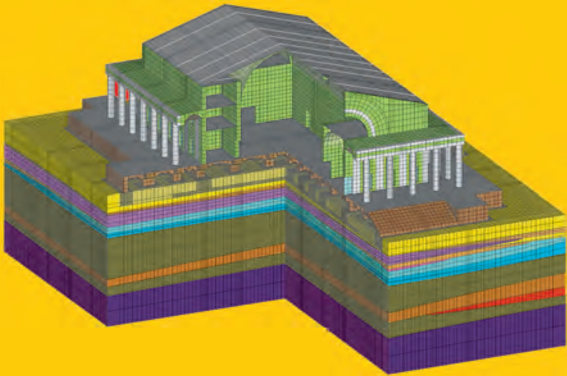

V.M. Ulitsky
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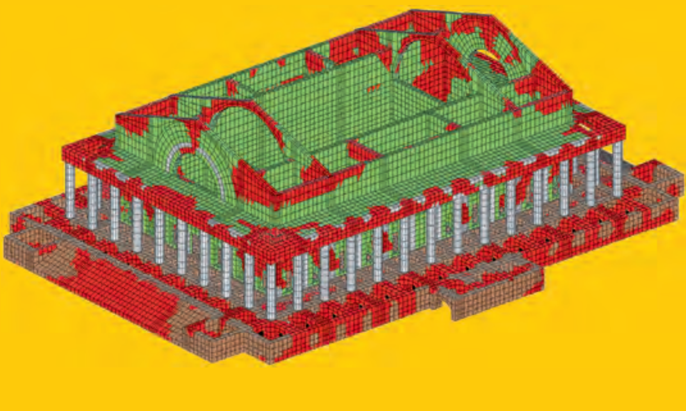
The Geotechnical Guidebook

a guide through foundations, subsoils and underground structures

second enlarged edition



Saint Petersburg
2013

V.M. Ulitsky
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(a guide through foundations, subsoils
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This second edition, enlarged and amended, presents an easily readable exposition of the major branches of contemporary geotechnical engineering, the science ensuring safety of construction, especially in complicated soils, prevalent in the Northern Capital*. The authors deliver fundamental information about peculiarities undoubtedly lying in wait for all participants of creation and realization of any construction project. Examples and illustrations are given, which demonstrate that underestimation of soil properties and erroneous geotechnical analyses are more often than not conducive to failures and significant financial loss, as well as to lack of clarity in terms of return period for the often considerable money invested.

* – a popular sobriquet for St. Petersburg. [translator’s note]

Reviewed by Prof. Dr. V.N. Paramonov, PhD CEeng (St. Petersburg Transport University), Dr. M.B. Lisyuk, PhD (ISSMGE).

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Introduction,
from which the reader finds out,
why he needs this book

"If you can't explain it simply, you don't understand it well enough."
Albert Einstein

Dear Readers!

Geotechnical Engineering is the subject of many clever books. They are written mainly for geotechnical engineers. What we would like to do, however, is to speak about this complex area of research, design and construction to a wider audience, which may comprise architects, clients, and investors.

Usually an investor can rather precisely appreciate complexity of a construction project – from structural design to superficial finishing. The difficulty mostly arises concerning the costs of the underground part of a building. Especially so, if the building is to be constructed not on the rather welcoming soils of Moscow, London or Paris, but on the soft yielding soils of St. Petersburg or Amsterdam. The problem becomes even more complex, if the project is located in congested urban areas, where it is necessary not only to build something, but also not to damage something which already exists in adjacency.

It should be noted at this point that here you will not find any recipes for replenishing your capital. It is not up to us, geotechnical engineers, to advise you in this area. But we could be rather useful to you in how it is possible not to lose money in building foundations and underground structures.

You know that you may have to pay very dearly for an error in defining the cost of construction of the underground part of a building. Such an error is capable of crossing out your entire business-plan and rendering your project unprofitable.

Badly arranged foundations can lead to catastrophic results.
And it is not important what you are building – a skyscraper or a small cottage.

Significant losses can result also from delays related to wrong organization of site investigation and design. The experience gleaned in days of a building boom is not suitable for the time of crisis and the post-crisis period. Earlier, financial loss caused by administrative mistakes was not so dramatic. Now is the time to optimize expenses for the entire building process. Now is the time to rely only on professionals.

In this book we tried to state briefly the fundamentals of taking managerial decisions in the areas of site investigation, design and construction, as well as to describe the basic methods to quickly assess their quality. Here we have actively used the advanced international expertise, where any architectural fancies are in direct coordination with the geology of a chosen site. The authors of the present Guidebook are members of international working groups of geotechnical engineers and are familiar with international expertise not "by word of mouth". Together with our colleagues, we are ready to act as skilled pilots to guide your building business through the dangerous waters of St. Petersburg soils and ambiguous construction codes, and thus to guarantee success of the most complex projects.

Part One

A trip from Site Investigation to Design

Chapter 1, explaining what geotechnical engineering is and whether or not the investor needs it

A well-known Swedish geotechnical engineer professor Sven Hansbo, during the opening ceremony of an international conference on geotechnical engineering held in Hamburg, with two hundred international specialists in the audience, theatrically locked the door and uttered in an eerie and mysterious whisper:

– Attention! We have a mafia in the audience.

The audience reacted with agitation.

– This mafia is us, – he continued. – Only we can bury any money in the ground and nobody would be any the wiser.

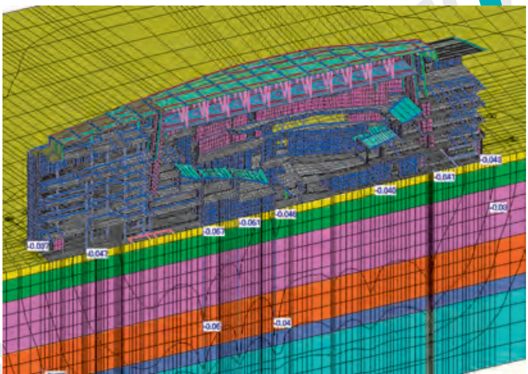
Dear Reader, Professor Hansbo was certainly only joking. It happened to be just a lecturer's trick designed to stir the audience. And the geotechnical engineer is never a mafioso. He or she is more of a pilot guiding the building business between Scylla of unprofitability and Charybdis of danger to the public.

Geotechnical engineering is a branch of building activity connected with soils. In our era of division of labour when each part of the human body is treated by a specially designated doctor, but the patient as a whole is interesting to no one and is in no one's interest, geotechnical engineering represents a synthesizing scientific discipline. It unites engineering geology, engaged in research of soils, soil mechanics, which creates numerical profiles, design of foundations and underground structures (with an understanding of all peculiarities of such design), technology of underground construction works, and, finally, control of ongoing works.

Geotechnical engineering is a synthesis

Engineering geology: *in situ* and laboratory testing

Soil mechanics: advanced numerical calculations and design



State-of-the-art working technologies

Only a synthesis of these related subjects known as "geotechnical engineering" is capable of providing to the investor a required result: to construct reliable foundations and underground sections, to top them with a superstructure and to keep existing buildings intact.

Geotechnical engineering is a science of managing construction-related risks.

In fact, geotechnical engineering is a science of managing construction-related risks.

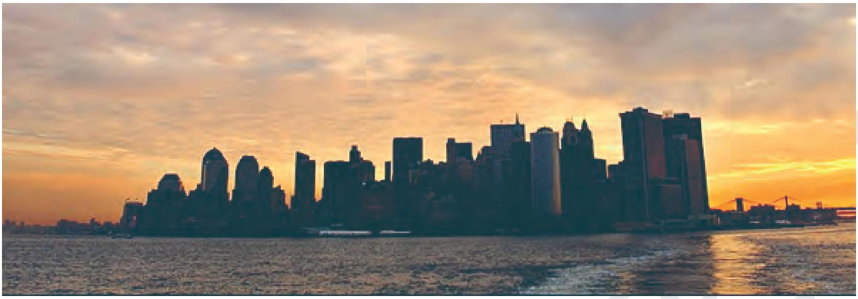
But it is absolutely not enough simply to call oneself a geotechnical engineer in order to be one. In getting oriented as far as the choice of geotechnical experts is concerned the investor can be guided by the international system which unites all representatives of the geotechnical profession in the world. The existing hierarchy is described below. There exists the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) whose President until the end of 2013 is Professor Jean Louis Briaud (USA). ISSMGE incorporates affiliated national geotechnical societies, including the Russian National Society on Soil Mechanics, Geotechnical Engineering and Foundation Construction (ROMGGiF), which has 50 regional branches. Out of those 50 branches the Northwest Branch headed by professor V.M. Ulitsky is second in number only to the Moscow Branch.

ISSMGE has a number of specialized Technical Committees engaged into various aspects of geotechnical activity. Russia is the host-country of one of them – TC207 "Soil-Structure Interaction and Retaining Walls" – managed by professor V.M. Ulitsky, Dr. K.G. Shashkin and Dr. M.B. Lisyuk.

Being entrusted with managing TC207 is a matter of professional pride, as it is a token of international recognition of the merits and achievements of the geotechnical school headed by professor V.M. Ulitsky.

Throughout the world no investor would be so light-headed as to call for solution of geotechnical problems a person who is not a member of ISSMGE. Membership in ISSMGE is an attribute of belonging to the geotechnical speciality, although, it certainly is not an assurance of a member's proficiency. It is better, if the organization which the investor decides to contact, is a collective member of both the national and the international geotechnical society (in Russia now only NIIOSP of N.M. Gersevanov in Moscow and "Georeconstruction" in Petersburg belong to both societies). Certainly, being a member is honourable and necessary, but is not in itself sufficient.

It is important that the person or the organization which the investor is going to work with can be represented by at least a couple of projects whose complexity is comparable with the level of tasks the investor needs to have accomplished. Here, if you are an investor, do not hesitate to ascertain what relation the person or the organization actually has to the projects they boast of. For, as is well known, victory always has multiple parents, and only defeat is an orphan.



Why are New York and St.Petersburg so different to each other?
The answer to this riddle can only be given by a GEOTECHNICAL ENGINEER.
Everything is explained by the triad: engineering geology, soil mechanics and geotechnologies.

Geotechnical Engineering is an exact science.

Some people consider geotechnical engineering to be a form of art. And if so, then as many experts, so many opinions. Such people believe soil to be an obscure and unknowable matter. Dear Investor, be wary of such "experts" even if they are members of the relevant society. Geotechnical engineering is, first and foremost, an exact science. Mastering it, certainly, demands great skills, profound understanding of soils, and being able to build numerical profiles that adequately represent soil behaviour. A contemporary geotechnical engineer is the one who perfectly knows behaviour of soil as a complex natural medium and is capable of an exact numerical prediction of its work in the subsoil of a building or a structure.

So, we began our exposition with the question who a geotechnical engineer is and where he can be found. Now we shall set in front of him the first problem.

Chapter 2,
explaining a preliminary geotechnical assessment of investment appeal of the construction project

How "dear" can our house be to us? It is the first idea that comes to mind of any investor. Who could answer such a question? It would seem, the answer is simple: the one who will build it, *i.e.* the Contractor.

The purpose of any red-blooded contractor is to sell what he has, and to spend as much of the investor's money as possible. It is the same as seeking medical assistance from a chemist: you will doubtless be sold the most expensive medicine that they have. Some investors go to several contractors, comparing their prices. This move is also totally wrong. One will offer you "a jollop for your dodgy tum", another "for your splitting head". Is it not far more reasonable to have your trouble properly diagnosed first, identify the problem, and only then prescribe treatment? In simple terms you need to see a doctor who is not a part-time chemist as well.

In our field of knowledge such doctor is an independent designer – the geotechnical engineer.

Let us assume you need to construct a house with two underground floors in a city centre. Adjoining your site are historic palazzos inhabited by posh bourgeois philistines (penniless old ladies, the erstwhile property owners, having been long moved away to council estates). If you contact the contractors, then one will offer you to drive sheet piles, another will declare that your condition can only be helped by secant bored piles, and the third will clamour for the use of jet grouting as your only succour. As a result you will be faced with a necessity to compare costs of three absolutely incomparable offers. And it is not until much later that you will realize that none of the three was any good. Depending on when the latter sad fact comes to light, it will be necessary to spend again – both money and time – on the new design, new site investigation, and in some cases on strengthening of the adjoining palazzos or – God Forbid! – on evacuation of the posh bourgeois philistines.

This, unfortunately, is not a dark fantasy but a very possible grim reality. To make sure it does not happen to you is easy – you need to contact a specialist right from the start. The very first architectural sketch should be shown to a geotechnical engineer of whom you need to inquire what costs he can envisage for that architectural *fata morgana*. Having performed the requisite calculations he then will be in the position to tell you how to construct one, two or three underground floors, what will have to be done to make it happen and what it will cost according to the average market prices. The geotechnical engineer will consider all possible construction options, will rule out the dangerous ones and will select several acceptable and feasible possibilities that can really be carried out by a number of known contractors. From that point on you will find yourself in possession of a requisite prescription and no chemist will be able to flog you an expensive and useless jollop.

> **To commission your design from a contractor is the same as to seek medical care from a chemist. You will be sold the most expensive medicine available.**

Possibly you are curious as to how it is possible to provide a set of preliminary geotechnical calculations having neither information on geology nor design, but only a general concept? There is nothing difficult here – there are archive data, the geological map of the city, and the loads which your building will be likely to generate are also quite clear to a specialist ("*there needs no ghosts, my lord, come from the grave, to tell us this*"). The assessment will certainly be a preliminary one but it will definitely suffice for the purposes of investment planning, or as they say these days, "feasibility study".

The geotechnical engineer will detail your programmes for site investigation and condition surveying of existing buildings, as he will also point out the major challenges you will need to tackle at the design stage. These will minimize your risks and optimize your costs, which, without the geotechnical assistance, may transform into a black hole entirely devouring profitability of your project.

As part of preliminary evaluation the geotechnical engineer will devise the site investigation programme, define the risks, suggest various possible solutions and evaluate the costs.

Thus, even at this preliminary stage we create the logic of successful future design and construction: we know what we are building, what site investigation we need for that and which geotechnologies will be required to minimize the risks in our specific situation.

Chapter 3,
explaining site investigation

Site investigation is often performed at the early stage when the investor is just considering plot acquisition for subsequent construction of his buildings. Therefore, if he is not willing to spend money prior to laying his hands on signed conveyance papers, no one can blame him. However, the investor must not forget that site investigation provides an information basis for taking all sorts of design solutions. The more vague and fuzzy the site investigation results, the greater the safety factors input by a reasonable designer. If the designer happens to be unreasonable, he will inevitably adopt erroneous solutions based on incomplete or unrealistic site investigation results and that is just another wording for “asking for trouble”. So, trying to save money on site investigation will always turn out to have generated losses several factors of magnitude greater.

At the heart of the construction risk management theory developed in the world today there is the following assumption – if site investigation with definition of the principal soil parameters was performed on site, the safety factor for foundation design is assumed to be 1.2. If there was no site investigation the safety factor may jump to as high as 4.0. Then it is up to you to decide what is less expensive: to conduct reasonable site investigation of to use three times the amount of reinforced concrete for your foundation.

Site investigation is the information base for adopting design solutions.

So how then do you distinguish a dedicated, conscientious and honest site investigation from a Mickey Mouse apology for the same?

The quality of site investigation can be easily assessed even by a layman. If you have listened to our advice in Chapter 2, then at this point you will have had a site investigation programme, drafted by an experienced geotechnical engineer.

In this case it is sufficient merely to compare the scope of the works carried out by your site investigation contractor with the scope specified in the programme. The geotechnical engineer would usually not say "no" to a request of reviewing the submitted site investigation results.

> **The more vague and fuzzy the site investigation results, the greater factors of safety assumed in the design.**

These days many a prospecting company (those responsible for site investigations) do not really like to do site investigations. What they do is drill holes, sample soils, define *in situ* density and water content, and build up geological profiles. And then they adopt the mechanical properties of soils (*i.e.* the ones used in calculations) based on SP, SNiP, TSN or whichever other codes may be at hand. Indeed, the SNiPs contain comprehensive information on soil properties throughout our vast country from Brest to Vladivostok. But these data only have reference value, as do the tables of local St. Petersburg TSN. They are good, in the best of cases, for a very very *very* preliminary assessment of the geotechnical situation. The authors of those tables would not have dreamt it even in their wildest dreams that their reference values would find their way into site investigation reports where investigators would proudly state that the primarily important values of E, c, *and* φ "are given as found in SNiP and TSN". One really should ask such apologies for site investigators: "What did you have to drill your holes for, if you failed to identify soil properties necessary for a safe design?"

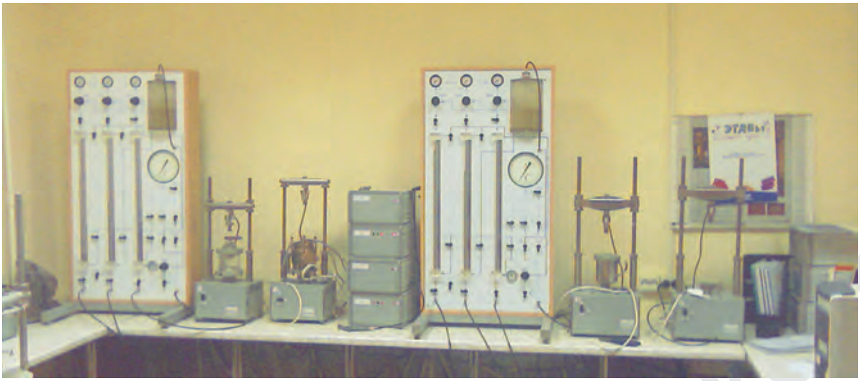
Dear manager, you do not need such investigations, even if they were carried out by companies famous in the past; do not waste your money on them. A true site investigation report **must be a thick one**. It must contain many plots, curves and graphs showing all manner of testing results. Indeed, in order to establish just one parameter of mechanical soil properties in only one stratum a minimum of six tests is necessary. If there are five strata in the geological profile, then the overall number of mechanical tests will reach one hundred. All of them should be appended to the site investigation report as plots showing testing results.

Those "illustrative materials" are used by experienced computer analysts to adopt a more justified and more economical solution. Which means, of course, saving money and reducing the time of works.

The present writers have over many long years campaigned for the betterment of site investigation quality. The long struggle came to a victorious end when St. Petersburg regional geotechnical codes ref. TSN 50-302-2004 were officially adopted, containing requirements that mandatory test of direct mechanical properties of soil be carried out. Today the main clauses of those TSN have been included into updated SNiP codes "Foundations of buildings and structures" and "Piled foundations" (the present writers acted as referees of the new codes for the Ministry of Regional Development of the Russian Federation).

So the acceptance procedure for a site investigation report can be really simple. You measure the thickness of the printed volume. If it is less than 1 cm the report is most probably of low quality. Then you compare the content with Appendix M of the codes (SP 11-105). This appendix is **mandatory**. It details what kinds of tests of what kinds of soils should have been done. It is to be remembered that St. Petersburg soils are of the highest (third) degree of complexity. For clay soils, which will always be found in the profile, there should always be compression, shear and triaxial tests. If such testing logs are missing in the report it means that the geologists had done only half the job, and had put in fanciful soil parameters.

Saving money on site investigation will be reflected in immense financial loss during construction.



A good laboratory equipped with state-of-the-art triaxial cells is a necessary condition for high quality site investigation results.

It needs to be said at this point that site investigation situation in St. Petersburg is not all too bad. New up-to-date automated laboratory complexes equipped with tri-axial cells started to appear. Progressive European static probing technology known as Cone Penetration Test (CPT) has been locally used for more than 10 years already. In conjunction with direct laboratory tests CPT is quite instrumental in obtaining realistic data on subsoils. For over 20 years, for direct soil investigation underneath foundation footings of historic buildings we have been using a technology analogous to Swedish dynamic sounding.

The instance of its first use in St. Petersburg by one of the present writers was not without a degree of healthy humour. Administrative officials for construction of that time went on the rampage and categorically demanded that Swedish probes must not enter the soils of the Soviet Union... But where are those officials now? And the dynamic sounding became commonly accepted for use in investigating subsoils of historic buildings. Even specialist codes for dynamic sounding were developed.

So, we can sum up **the main features of a good quality site investigation** as follows:

- Direct tests conducted to establish mechanical soil properties;
- Triaxial tests;
- State-of-the-art cone penetration tests (CPT) or dynamic sounding, if investigation is to be conducted underneath existing foundations.

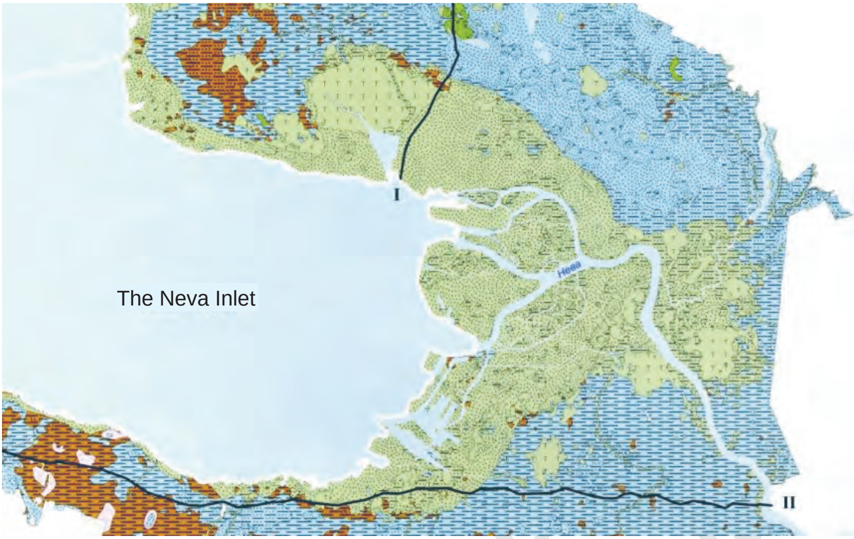
Both scope and volume of site investigation should be in strict compliance with site investigation standard procedure requirements, updated national codes of practice and local codes.

To conclude the present chapter we shall suggest **a simple way of evaluating complexity of site geology**.

Our local soils are like a layered cake. This cake's composition is not identical for all city areas.

Peter the Great was obviously not only a great tsar, but "a great geotechnical engineer" as well. The boundaries of the historic city centre are, quite curiously, the same as those delineating the area of soft soil distribution. Out of all council housing areas only the Lakhta and Komendantsky Prospekt can rival the historic centre as far as the complicated geology is concerned. Therefore, the people of the geotechnical profession in our city are in high demand. It was in St. Petersburg that the professional Russian geotechnical school was born. It happened in the Transport University in the end of the 19-th century. This school's traditions are being preserved and replenished still.

For the city centre the typical composition of the "soil cake" is as described below.



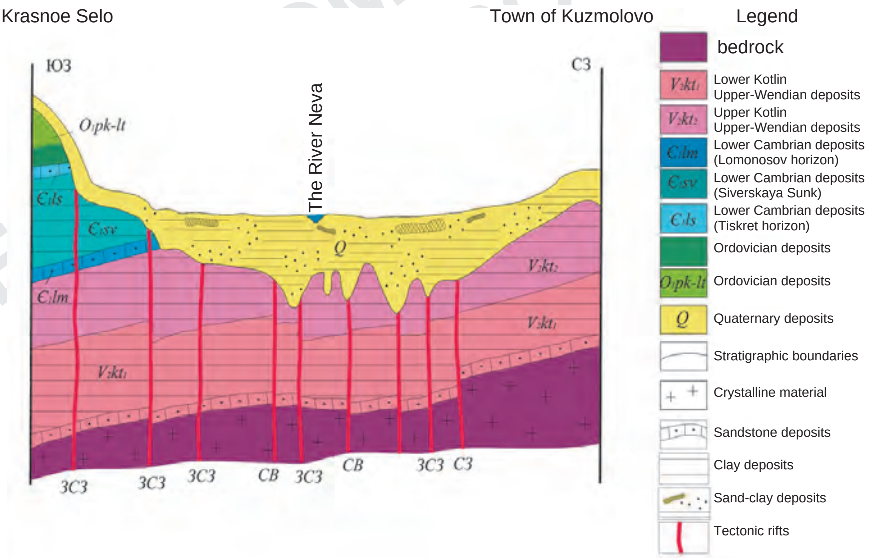
An engineering map of quaternary deposits in St. Petersburg (Source: Geological Atlas of St. Petersburg): the area of soft soil distribution is indicated in green.

On top there are fills up to 3 m thick and underlying the fills – 2-4 m of sands. It was on these sands that the historic St. Petersburg was constructed. Below are the well-known St. Petersburg soft clays. They have a particularly repulsive character: under any influence they tend to turn into viscous swill. It is that kind of soil in which excavators sink. Even human beings get well stuck in. This material gets heaped up into a lorry and starts spilling over the brim of the skip by the time the lorry reaches the site gate. Even tips and recycle centres refuse to accept it. They say: before we can accept this soil please remove the water from it. And how on earth is it possible to remove it if 10 thousand years have elapsed and it is still there?

St. Petersburg soft soils tend to turn into viscous swill under any influence.

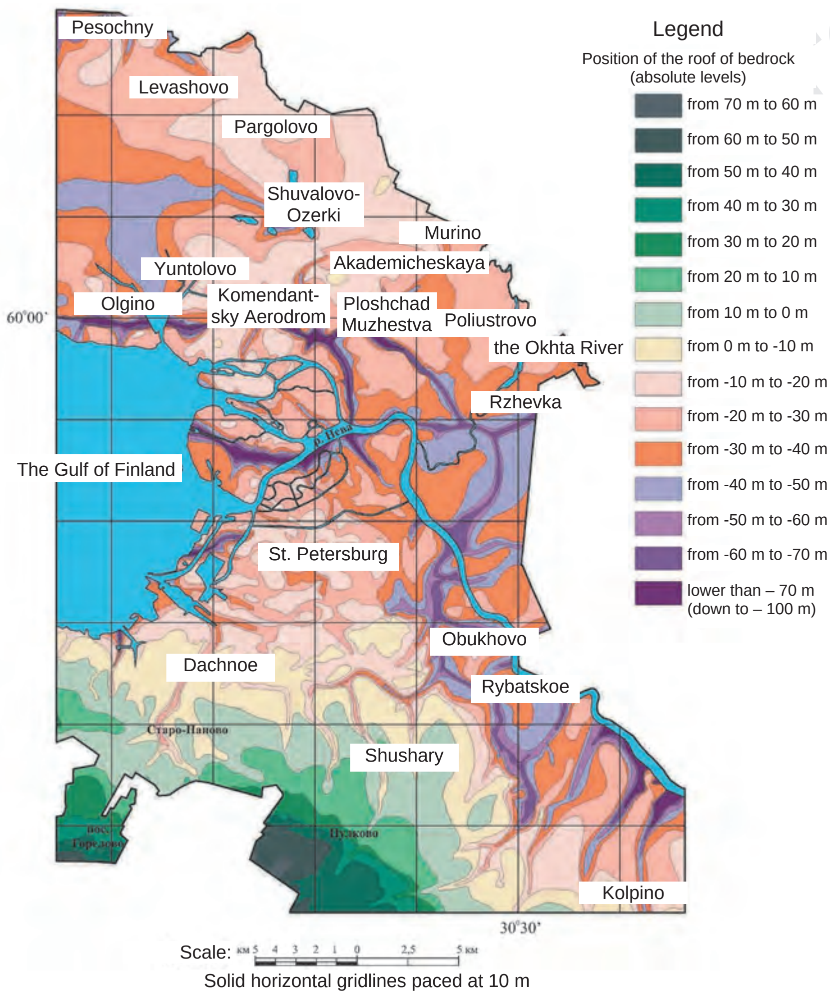
During bulk excavation in order not to sink the excavator moves along a pontoon constructed of metal pipes 800 mm in diameter.

More or less decent soils begin only from the depth of 20 m. They are usually referred to as "moraine". There is an erroneous belief that moraine is always a reliable soil. Unfortunately, this assumption is far from the truth. Moraine happens to considerably vary in quality. Sometimes it is not very much different from soft soils (for example on Vasilevsky Island). One therefore should treat it with caution.



A schematic geological stratification profile of St. Petersburg with elements of tectonics (according to E.K. Melnikov).

Really reliable soils in the center of St. Petersburg are encountered at depths of more than 30-40 m. They are commonly referred to as Wendian deposits or Proterozoic clays. It is in these deposits that St. Petersburg metro lines are constructed. In some places prehistoric rivers had cut deep beds (paleovalleys) at depths exceeding 100 m.



Position of the roof of bedrock of sedimentary cover in the territory of St. Petersburg (data from the Geological Atlas of St. Petersburg, 2009, reinterpreted and amended by Prof. R.E. Dashko (2011))

One of the paleovalleys stretches approximately from the underground station "Primorskaya" down to the emblematic Bronze Horseman sculpture. Remember the famous stanza from Alexander Pushkin's eponymous poem:

"O, karma's mighty sovereign!
Not thus you'd reared Russia, sullen,
Into the height, with a curb, iron,
Before an abyss in your reign?"

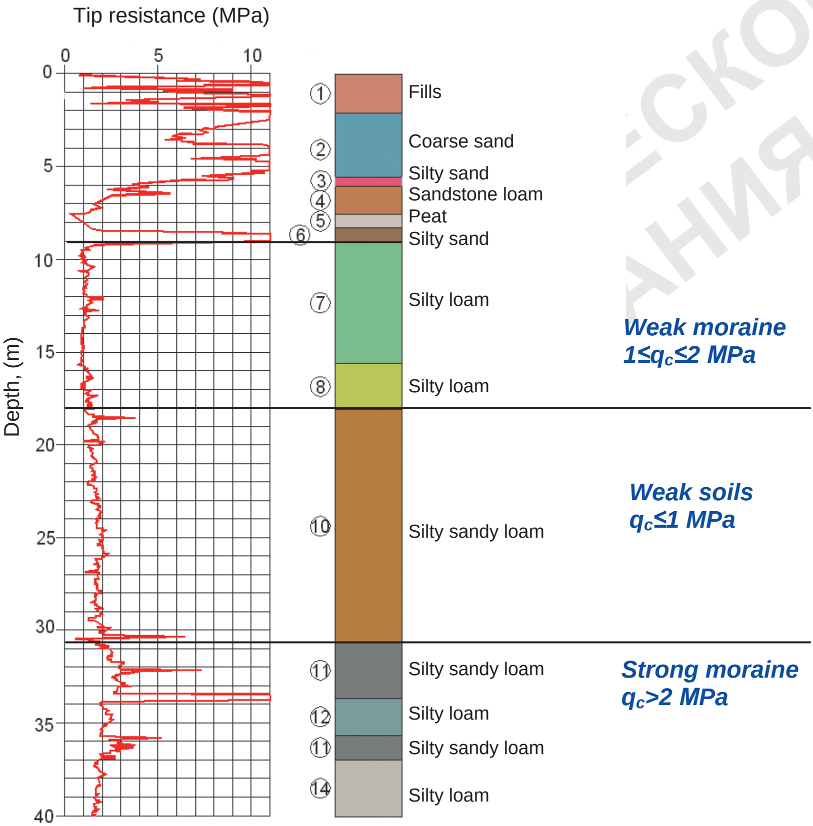
(Alexander Pushkin "The Bronze Horseman",
translated by © Yevgeny Bonver)

The italicized phrase has a specific geological implication. Another paleovalley crosses the metro line "Lesnaya" – "Ploshchad Muzhestva". This so-called "washout" is no washout at all and there is no underground river to be seen. There used to be a river but millions of years ago. The prehistoric riverbed is filled with very dense water-saturated sands which turn into quicksand if released into a tunnel.

Below the Wendian clays at depths of 200…250 m there lie rocky soils – granite and granite-gneiss. This rock plate, like any other, is split through with a series of tectonic rifts. Some charlatans scare off the punters by the intimidating fact that their house stands on a "tectonic rift". But in reality one should not fear this "terrifying" word. The house really needs to be the central character from Hans Christian Andersen's "The Princess and the Pea" to be able to feel presence of a crack in the bedrock through a 200-meter "feather-bed" of sedimentary soils.

As we bring to conclusion this brief travel into the deep we shall answer, at last, the question put forward in its beginning. So, there, you have a volume of a site investigation report sitting on the desk in front of you. **How to appraise "the degree of disaster",** ***i.e.*** **how complicated are the geological conditions of your site?** The most simple and objective way is to look at Static Penetration (CPT) graphs. The essence of this test is simple: a probe constructed of a conic tip on a steel bar is pressed into the soil, whereat the resistance of the soil to penetration is measured. It is the tip resistance that we try to evaluate looking at the graph. If this resistance is around or lower than 1 MPa you have got a layer of weak soil.

Usually they are sandy loams (sand with clay) and loams (clay with sand), either lacustrine-marine (designated as *ml* in soil profiles), or lacustrine-glacial (designated as *lg*). Geologists also use the letter *g* to designate moraine. If tip resistance for moraine is lower than 2 MPa the stratum is rather weak. If the value is anything in the order of 2 to 4 MPa it means you are in luck.



Example of defining geological complexity of site conditions based on Cone Penetration Test results.

The cone tip usually cannot enter the strata of Wendian deposits. If, nonetheless, it has entered, there is something wrong with the stratum. Most probably it is the so called "upper dislocated zone", which was spoiled in the prehistoric time, when the Wendian strata were on the surface.

Outside the historic centre of St. Petersburg it is possible to encounter essentially more favourable geological conditions. Sometimes the moraine is closer to the surface, with firm clay underlying it at higher depths. In the vicinity of Poklonnaya Mountain there is an area of thick deposits of strong sands.

In any case, with the static penetration test you can, as if using a crow-bar, probe your "soil-cake" and define if any weak strata are present.

Chapter 4,
explaining advantages of building standards

Until recently neither the necessity nor the advantages of building standardization have been challenged by anyone. The confusion began, probably, with the conversations about Russia's joining the World Trade Organization. It is obvious, that membership in the WTO also assumes creation of uniform game rules on the building market.

As is well known, construction activity in our country had always been regulated by various codes, such as SNiP, SP, TSN, GOST, *etc*. In this respect our country was no exception to the general rule, but fully corresponded to approaches adopted in other developed countries. In Germany there is a system of DIN standards (which had at one point served as a prototype for our SNiP), in the UK – the British Standards, in France – Standards of the French Republic.

More than 15 years of laborious work were required for experts from the countries of the European Union (complemented by serious state financing) to have developed and coordinated uniform European normative system of Eurocodes. And each Eurocode, including Eurocode 7 ("Geotechnical design"), consists of a general part common for all European countries and a National Appendix. Eurocodes are new generation norms. Whereas the older national norms (both the German DIN, and the Soviet SNiP) often contained fine and minute regulations on HOW it is necessary to solve this or that building problem, Eurocodes pay attention to WHAT is necessary to solve. The matter of "HOW?" remains in the domain of the specialist.

It is necessary to say, that the hearsay about progressiveness of Eurocodes is a little bit exaggerated. As a result of a set of compromises on the part of various European geotechnical schools in the general part of Eurocode 7 not too many specific requirements have remained. Each and every specificity has been "exiled" into the National Appendices.

Thus, Eurocodes are a recommendatory, and not, strictly speaking, obligatory document.

Eurocodes pay attention to WHAT is necessary to solve. The question "HOW?" remains in the domain of the specialist.

However, it needs to be understood and remembered that in case of failure or an accident the Public Prosecutor will by all means express interest as to how strictly you had been following Eurocodes. If you had been following them altogether not too strictly, the Prosecutor will continue his line of inquiry asking if you might have been conducting your personal profound research into the subject in question? If the answer to this is still a "no", then the final question will sound thus: "In this case why had you not been following Eurocodes?"

In a situation like this it will take only the bravest, the most independent and the most professional of all researchers not to follow Eurocodes.

In Europe, by the way, for complex projects there exists such a useful thing as "the four eyes rule". It means that one pair of eyes belongs to the designer, and the other to an independent expert hired by the municipality using the funds that have to be necessarily provided by the Investor. It means that decent money is used to hire an expert, having authority, standing and reputation not inferior to those of the designer. Such a specialist is capable of delivering an analysis even deeper than what (as we are all used to) is usually delivered by the State Expert Board. He can independently repeat the most complex calculations, check up constructive schemes, units, *etc.*

The next barrier against non-professionalism in the leading European countries is the obligatory insurance. It is beyond all reasonable doubt that the insurance company, which is liable with its own purse for the quality of the design product, will apply its best effort to ensure correctness of the design to avoid onset of "an insurance case". The insurance sum depends on the quality and profundity of all design stages.

Unfortunately, the Russian practice of construction insurance is not deprived of its remarkable national features. For example, one respectable natural monopoly, employing a veritable army of lawyers, had its building insured. At an hour of doom one of the building's sections collapsed and the owners asked us to establish which of the cases described in the insurance policy had taken place. The insurance agreement looked very decent indeed. It dealt in every detail with such noble issues as the insurance cost, and even in more than every detail with the most important and fascinating subject – the procedure of payment of the insurance premium (the lawyers had tried hard to show what they are worth). It was also mentioned that all the tedious and uninteresting stuff was contained in the Insurance Rules appended to the contract. In these rules, in the smallest of all readable prints, it was written that errors on the part of designers and contractors were NOT to be construed as insurance cases, and neither were any cracks appearing as a result of the building's deformations. A collision with an aircraft was equally dismissed as a potential insurance case, for, certainly, the subject matter of the agreement was insurance against *construction*-related risks. Eventually, we completely failed to determine what should have happened to the building for the victims to be in the position to receive any money at all. Indeed, there exists no reasonable idea that may not be bungled by its "skilful" practical implementation.

Finally, the most effective tool to maintain professionalism in the area of construction is the educational system to prepare and control experts, accepted, for example, in the UK and the USA.

A graduate of a British civil engineering college receives the certificate not from his *alma mater* but from a society uniting all civil engineers of the country, affiliated with a London-based high school known as the Institution of Civil Engineers. A specialist who wishes to grow in his career needs from time to time to sit examinations there and acquires first the right to put his signature on drawings, then the right to manage a team of specialists, further on – the right to act as a design evaluator. Any failure on the part of such a specialist is immediately made public and may lead to disqualification with a lowering of status or even expulsion from the Institution. If the latter is the case the poor fellow would have to look for another occupation altogether. Even colleges and institutes may get blacklisted if they have bad statistics in terms of incidence of errors amongst their graduates and retrained specialists.

When you get acquainted with the Russian reform of construction codes you understand that reformers must have heard something about that European system. But, unfortunately, many of Russian reforms go by the principle expressed in the line of the Internationale: "*Of the past let us make a clean slate …*" Experts are not involved in process of reforming. As a result we risk to transform a rather well regulated building industry into a field of activity dangerous to people.

> **There has been no historical precedent for high-rise buildings or underground structures to have been built without a scientific input.**

Before the October Revolution in Russia, there were institutes of civil and highway engineers not less respectable than the analogous British ones. Would it not be more reasonable to revive those honourable structures than to breed the so-called "self-harmonizing independent trade entities" (SHITE) which unite (who would have thunk?!) no fewer than 50 companies. No norms, codes, regulations or rules will ever arise in the midst of those "gentlemen's clubs". No administrative structure will ever be able to cope with reforming of the building norms without participation of professionals. One should not really be labouring under the illusion that the SHITE (just consider the phonetic implications of this abbreviation!) will ever be capable of creating any codes without state financing. These are dangerous phantasms. In no countries of the world has science ever developed without support of the state. And without science there will be no codes, regulations or norms of any kind. There has been no historical precedent for high-rise buildings or underground structures to have been built without a scientific input of the highest order.

In this respect to give full control of building normalization to freshly formed entities without a scientific input is simply dangerous. By the way, science cannot come in various quality grades, like sturgeon of the theatre canteen manager in Mikhail Bulgakov's “The Master and Margarita”. It can only be of the premium grade – the rest is pseudoscience.

A successful attempt at creation of technical rules meeting requirements of “*these our times*” are the St. Petersburg “Codes of Foundation Construction” (TSN 50-302-2004). They are written in the European fashion. They detail what must be done, and the question HOW it must be done is left in the hands of professionals. Actually, these codes can act as a certain safeguard for Mickey Mouse workmanship. On the other hand, they have facilitated consideration of design documents both for the customer, and the State Expert Board, as they contain clear qualitative and quantitative criteria of what the designer must perform. Today the substantive provisions of the TSN 50-302-2004 have been included into the updated edition of the SP 22.13330.2011 “Foundations of Buildings and Structures”.


Система нормативных документов в строительстве

ТЕРРИТОРИАЛЬНЫЕ СТРОИТЕЛЬНЫЕ НОРМЫ

ПРОЕКТИРОВАНИЕ ФУНДАМЕНТОВ ЗДАНИЙ И СООРУЖЕНИЙ В САНКТ-ПЕТЕРБУРГЕ

ТСН 50-302-2004 Санкт-Петербург

ИЗДАНИЕ ОФИЦИАЛЬНОЕ

Администрация Санкт-Петербурга
Санкт-Петербург
2004

What then is it possible to advise to the manager concerning the norms, standards, codes, regulations and the like? Forget the talks that the authority of the SNiP and the TSN have been lifted or have come to possess merely recommendatory nature, as this will never be an opinion of any expert board, governmental or otherwise. The board will ask you: “What do you have to substitute for those codes?” and add: “If you do not like the codes, write new technical regulations, have them approved by the government and then, as they say, Robert's Your Father's Brother, we shall have it examined. If you are having second thoughts as to the writing of new regulations, then work in conformance with the currently applicable ones.

Forget the talks that the SNiP and the TSN only have merely recommendatory nature.

Recently in the power circles a curious idea took root – to help the developer by removing some bureaucratic barriers, including even (O, how terrible to behold!) the necessity to have one's design approved. However, instantly a proviso is introduced to the effect that this liberty will be immediately curbed by means of increasing the developer's liability. Differently put: "Mr. Developer, you complain that the state has got you with both your hands tied behind you? We shall assist you thus: from now on you will be happy to tie your own hands for your own safety and benefit".

So, the developer's best bet is to write down in the contract with surveyors, designers or contractors a treasured phrase "all works are to be carried out according to the SNiP, the SP and the TSN", whereupon the list of the latter should be included. This way you can return to a regulated and a manageable situation and safeguard yourself against every possible speculation.

It is most important to have such a clause included into your contract if you happen to be dealing with foreign designers. You can believe our wide experience: no foreign designer will be able to cope with his task without an adaptation to the Russian reality – any design executed by western standards will not be approved and as such will be useless on a building site.

Chapter 5.
How shall we build – legally or “reasonably”?

As is well known, nature has never forgiven violation of its laws. We live in the period when satellites fall, nuclear energy units fail and houses crack, even those that have been recently constructed. Thus gradually we are moving to a dangerous line – a period of anthropogenic catastrophes. The level of design reliability plummets, durability of new buildings drops and lifetime of previously constructed ones declines because their maintenance is neglected, wear and tear disregarded and monitoring suspended. How can we battle with this abomination? Only relying on professionals.

The new Russian Federal Law “Technical Regulations on Safety of Buildings and Structures” (ref. 384-ФЗ) is likely to be of support. It has come to regulate the whole cycle of a building’s life – preliminary site investigation, design, construction, use, maintenance and even demolition. Its main effect is to safeguard against non-professionalism, mismanagement and incompetence, which safeguard is not always welcomed by everyone. It can be contended that these days, in order to tidy up the affairs in construction business it is sufficient merely to observe this law.

> **To clean things up in construction business it is sufficient to merely observe the FEDERAL LAW.**

The law has brought to an end the erstwhile speculations that construction law was no longer effective

The law puts down strict requirements to conform to those standards and codes of practice which may have an effect on public safety. The process of updating construction law has now been almost brought to an end, the present writers being active participants therein, working in a committee of the Ministry for Regional Development. Despite the very tight schedule, it has been possible to update the codes, which have been neglected for several decades, by means of introducing reasonable contemporary requirements.

> **The federal law has brought to an end recent speculations on non-obligatory nature of construction codes.**

The principle of limit states – the basis for safety

The concept of ensuring mechanical safety of a building realized in the law is well known to designers and has long been accepted both in Russia and elsewhere. This concept is quite simple: no limit states as regards stability and strength both in the subsoil and superstructure can be allowed to appear during construction or use of buildings, *i.e.* it is not permissible to generate dangerous deformations which will stand in the way of a building's health. And yet the fact that this concept has acquired the status of a law is hard to overestimate. It will allow to make responsible those designers who willingly or unwillingly due to their incompetence rob projects of their reliability. It will clear the air a bit in the design community by purging it of bounty hunters whose activities become ever so more dangerous every year.

Some individuals, who we may call apologies for designers, think that it is possible to bypass the codes only a little bit and everything will remain hunky dory. However, it is impossible to allow a limit state to appear for even 5 minutes, as it is impossible to rob a bank of a million for 5 minutes. Both deeds are equally criminal. The primary cause of any failure is always violation of codes, either during construction or use of a building.

> **It is impossible to allow a limit state to appear for even 5 minutes, as it is impossible to rob a bank of a million for 5 minutes. Both deeds are equally criminal.**

The law requires three-dimensional non-linear soil-structure calculations

For the first time on the level of a federal law requirements have been introduced concerning computational models, finite element profiles and modelling assumptions. First and foremost, they should reflect *actual* conditions of a building's behaviour, *i.e.* those that are actually in place. A model should always be practically verified. Nothing important should be forgotten in calculations.

The law states that it is required to consider *spatial behaviour* of structures, *geometric* and *physical non-linearity*, and even *plastic and rheological behaviour of soils and materials*. Soil, having been created by nature, has no manufacturing certificate, like concrete or steel.

Mechanical characteristics of soil change not only with depth and extension, but also during application of loads and during deformation. All this peculiar set of features is necessary to account for in calculations.

It will be interesting for the reader to know that *rheological* properties of soils (the way soil is deformed in time), accounting for which is required by the law, are impossible to define authentically either in a laboratory, or *in situ* (as geologists call the tests done outside the lab). We have spent twenty years to determine rheological properties of insidious St. Petersburg soils based on long-term field research. We do not hold this valuable information secret – everything is published. But here is the trouble: not a single computer program available on the market today is capable of considering rheological problems. It was due to this that we had to develop our own software complex *FEM models*, capable of solving all possible soil-related problems (by the way, today it is the only software package which is not only certified to conform to all soil standards, but also verified, *i.e.* checked on projects studded with instrumentation gauges all over).

The law also stipulates separately the necessity to calculate buildings and subsoils with allowance made for *their interaction.*

The law stipulates the necessity to calculate buildings and subsoils with allowance made for their interaction.

If the present writers had conceived to create such norms that no one apart from them could conform to, they would still have had scruples to include some of those positions that today are obligatory requirements of the Federal Law 384-ФЗ. Who, for example, is able to calculate interaction between superstructure and subsoil, especially in nonlinear setting? It can be done only by few teams in Russia and abroad.

The law requires for rheological soil properties to be accounted for in calculations.

Fortunately, we have been successfully engaged in such calculations for the last ten years (opportunities for combined calculations actually appeared owing to creation of *FEM models*); we constantly communicate with colleagues from Europe, the Americas, India, Japan and Australia, enriching each other's knowledge and experience. Why should we not be in the position to assist our Russian colleagues in the building trade? The Inquisitive Reader will learn about combined calculations if he or she comes to read Chapter 11 of this book.

The law contains a requirement on safety of existing buildings

Negative influence of construction on surrounding buildings should be as little as possible, no threat concerning life or health of civilians (and even animals, and plants), no danger to property or welfare is acceptable. This requirement has long been contained in Moscow and St. Petersburg geotechnical codes. Now, getting the high status of the federal law it will allow to increase safety of existing buildings.

Importance and timeliness of this requirement can be realized when you notice that these days more and more construction designs potentially leading to damage of adjoining buildings are submitted. And there, instead of cardinal revisions of design towards its increased safety, the client is offered to fork up on strengthening of an entire neighbourhood – sometimes within the zone of 50 m from the building he is about to construct!

According to the law, the design should stipulate the measures to reduce consequences of anthropogenic influence. Construction work in St. Petersburg should be conducted in such ways that do not lead to infringement of the natural structure of weak clay soils. What kind of ways they are the Inquisitive Reader can learn having read through Part Three of the present "Geotechnical Guidebook".

The law welcomes application of research results

The builders now face new challenges of the present day – construction of high-rise buildings, development of the underground space, and complex reconstruction of city quarters built on weak soils of St. Petersburg. Without scientific research, without generalization of achievements of modern geotechnical engineering, solution of these problems is absolutely impossible. The law requires scientific support of site investigation, condition surveying and design for high responsibility projects.

Safety measures should be supported by research, calculations, tests, modelling of dangerous influences, and relevant risk assessment. If the designer sees insufficiency of norms or standards, he or she should fill the blank by means of science. Unfortunately, it is only teams of most qualified designers, actively engaged in scientific research, that are capable of such a thing. It is rather pleasant that the law promotes involvement of research into practice of design, which will lead to progress in the entire building branch.

The law has established new requirements to carry out monitoring:

- Monitoring should trace conditions of subsoil, structures and engineering services and to ensure their conformity to the design;
- The design should envisage monitoring of all structures in the zone of construction influence;

- Monitoring should be conducted not only during construction, but also during use and maintenance of buildings.

The basis for monitoring of ultimate parameters should be the results of soil-structure interaction calculations. The issue of monitoring is dealt with in Chapter 16 of this “Geotechnical Guidebook”.

The law establishes strict requirements to independent expert assessment

According to the law an important element of an obligatory examination of buildings’ conformity to standards is expert assessment of site investigation results and design documents. It should establish their conformity to requirements of the law before the beginning of construction, including requirements to maintenance of mechanical safety. For modern design practice the key requirements are those in respect of numerical models.

Unfortunately, in the majority of cases both governmental and non-governmental bodies of experts are not in the position to estimate reliability of calculations and the numerical models used therein. Some even do not have such objectives. In this connection, in order to help experts, we have developed simple methods of assessing computation reliability, given in the Part Two of this “Geotechnical Guidebook”. Those methods will assist one in revealing bad calculative mistakes not resorting to duplication of calculations.

So, the new law, ref. 384-ФЗ has challenged the entire building community – researchers, designers, builders, maintenance organizations, expert supervisory bodies – to prove their professionalism in terms of whether they can conform to its exacting requirements. Methodical and obligatory application of the law will not leave any room for amateur formations which often win tenders conducted according to the principles of cheapness or personal proximity. Requirements of the law can only be met by a true professional, actively engaged in self-education, and for whom scientific research is food for thought. The law is capable of building innovative approaches to building process and of preserving the greatest property of people – the historic architectural heritage.

Chapter 6, explaining geotechnical substantiation

When one elderly professor, who had chosen a sideline career of a designer, was asked by the State Expert Board: "And what about your geotechnical substantiation?", he retorted with spirit: "Enough of your Ulitsky lark!"

> **Geotechnical substantiation is intended to choose and realize the optimal design solution.**

What is this "lark" – the geotechnical substantiation – and who apart from professor Ulitsky needs it?

As it is written in the TSN, "Geotechnical Substantiation is intended to choose the optimum design option and technology of its realization, providing reliability of a reconstructed or constructed building and safety of existing structures". When we formulated this thesis, it seemed to us, that it was mostly for the benefit of the investor. By the way, today this requirement is written down also in the new edition of the SP "Foundations and Subsoils of Buildings and Structures" (therein the geotechnical *substantiation* is referred to as the geotechnical *prediction* which does not alter its essence).

Really, how will the investor know that the designer has come up with the optimum design solution if no calculations for other possible options have been submitted? We reiterate here that geotechnical engineering is exact science; therefore each option can be subjected to strict calculative assessment. This calculative assessment allows to define both the deformation of the main structure and that of the existing buildings. It optimizes possible expenses as early as the pre-design stage and confirms them during design process.

We already mentioned geotechnical calculations in Chapter 2, dealing with tentative estimation of a geotechnical situation. The geotechnical substantiation repeats and deepens this estimation based on new, much wider spectrum of preliminary input parameters. We are already in possession of site investigation, condition survey reports for existing buildings, and architectural solutions. Now we can choose the optimum length and diameter of piles, pick up the method of stabilizing deep excavation cofferdam – from a diaphragm wall to a line of sheet piles, and define sparing geotechnology for specific construction conditions.

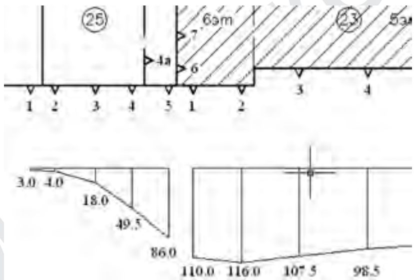
This choice is made on the basis of very precise criteria expressed in the codes. These are: the building's own deformations (settlement, settlement differentials, tilts) or permissible additional settlements of existing adjacent buildings. These requirements do not contradict domestic or European standards. Main principles of these specifications are not different. They are subsumed under the principle of calculating foundations jointly with superstructures based on two groups of limit sates: one for strength and stability (reliability), the other for deformability (serviceability).

There were times when we could build according to the principle: one house built, two destroyed. Those times are over.

Recently buildings and structures have been seldom built in greenfield conditions, *i.e.* when there are no structures and engineering services in the zone of possible risk. More often than not construction is conducted in congested environment of a densely populated city. In such cases the criterion of reliability for the building which is being constructed becomes secondary. The primary criterion becomes that of safety of existing buildings and communications. Very often settlements of 10, 15 and even 20 cm are permissible for the new structure. However, when it is erected in congested city conditions it becomes necessary to limit its settlements to values 5-10 times smaller so that existing buildings are not pulled into the settlement trough.

It is this numerical order of settlements (2-3 cm) which is permissible for ordinary historic buildings in the city of St. Petersburg.

Classical settlement related cracks which appeared in a house adjacent to a new building with wrongly constructed foundations. The new house (in this picture located on the right) developed settlements and "pulled" its neighbor into the trough. Below you see the settlement diagram (mm)



If you exceed the value of permissible additional settlements, adjacent buildings will develop dangerous cracks and the investor will face a time of legal proceedings against residents and the city administration, followed by payments of considerable additional funds to either restore the damaged houses or to evacuate the residents elsewhere.

There were times when one could build houses in historic city centres according to the principle: one house built, two destroyed. Those "good" old times are over. It was because the state was always there to make good the damaged houses. But these days every house or flat has an owner. Many flats have expensive furnishing in relation to which settlements of even 2-3 cm, safe for the building's structures, can appear unacceptable. (The present writers have acted as experts in litigations between aggrieved residents and developers).

The designer needs to devise such foundations that the new house develops minimum settlements and does not provoke settlements of adjacent buildings.

Besides, the designer needs to choose such earthworks technology which will not lead to settlements of existing buildings once construction works have commenced.

The usual designer will not be able to solve such problems for you. That is where the geotechnical engineer comes in. The situation when the designer and the geotechnical engineer work together is ideal (as it has developed in "Georeconstruction" Institute where a civil engineer always sidelines as a geotechnical specialist). It frequently allows to obtain essentially new, economically effective design solutions. It is much worse when those two professionals work separately and independently of each other. So, the geotechnical substantiations allows the manager to control the basic solutions of the designer, at least those that define the project cost.

If the geotechnical substantiation is missing, the manager lacks the tool for such control. How to distinguish a good and proper geotechnical substantiation from the Mickey Mouse produce, of which, unfortunately, there is a lot about?

It is formally possible to check up its structure and composition against the list detailed in the TSN. In fact, the requirements contained therein were created for all participants in the process.

Those who are interested can carry out their own estimation of the document's quality being guided by the simple principles advised in Part Two of the book. If you prefer to delegate this work to an expert let him be the one who will read this part, whereas here we would like to bring to your attention some simple and clear answers to the key questions contained in a geotechnical substantiation.

How to choose the type of foundation?

Building something in the green field, that is to say, outside the city or at the outskirts, where the distance to the nearest structures is more than 20 m, we choose such foundation type that the settlements of buildings do not exceed the permissible. For low-rise buildings (up to 5 floors) the usual shallow foundation (without piles) will most likely do. It is only necessary to take care that blocks of unequal height do not touch (as the higher block will by all means drag the lower one into the settlement trough). Differential settlements of a building can prove dangerous.

Certainly, it is necessary to glance into the report on geology. About the soil "cake" and about how it is possible to probe it by means of the cone penetration test we spoke in Chapter 3. If under the foundation footing there are no weak strata – a shallow foundation type can be chosen.

It happens sometimes that outside cities geological conditions are so favourable that shallow foundations can be used even for 16-storeyed buildings.

But more often than not high-rise buildings require piled foundations. They help reduce settlements of the building because they transfer the loads to stronger deposits of the subsoil.

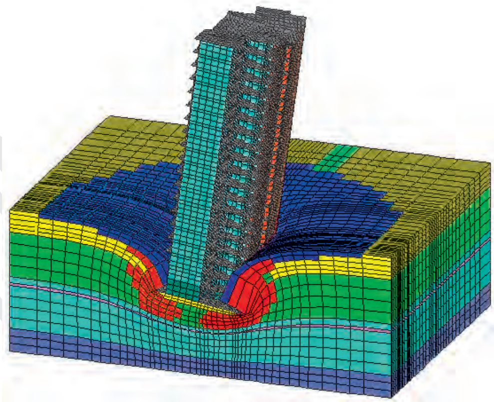
The settlement itself is not as terrible as its differential. Settlement differential causes a lengthened house to crack and a long vertical one to tilt.

Not all geotechnical mistakes look so attractive.

When building in congested city conditions, the type of foundation to be chosen is determined by potential settlement of existing buildings.

There was once a famous scandal with Shipkinsky Lane, 3 in St. Petersburg. Someone had designed an unpiled slab foundation for a 16-storied building. The tilt was first noticed by residents when they began to cast blinding and discovered a differential of 15 cm (!) within a single flat. Subsequently the lifts jammed. The building had tilted by 80 cm, which is visible even in the photo. The city invested a lot of effort and money to correct the tilt. The conclusion is simple – if you wish to save money on foundations, do not be thrifty with the geotechnical substantiation. The mistake with the choice of foundations would have become obvious already at the preliminary calculation stage.

The consequences of the erroneous foundation choice for Shipkinsky Lane, 3.



When building in congested city conditions you, most likely, will not be able to manage with just shallow foundations. Requirements to settlements become tougher and are defined by permissible settlements of existing buildings. Even for a 5-storied building you might require a piled foundation. And the foundation technology you choose should also not be detrimental to existing buildings. We shall speak about sparing geotechnologies below in Chapter 14.

What determines the necessary pile length?

The answer to this question is very simple: piles should be of such length that settlements of a building are permissible, allowing for it to be safely used. The quantity of piles and their arrangement are determined by bearing capacity of a single pile in soil.

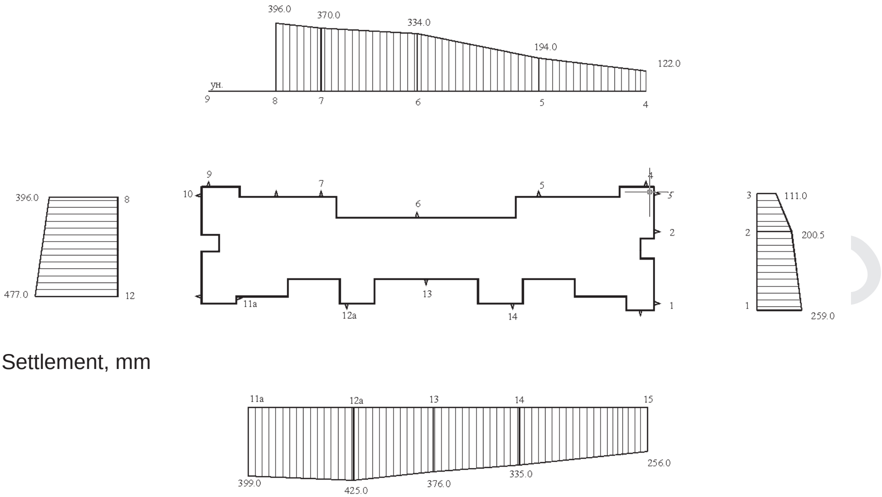
It is not worth it trying to use half-length and half-price piles. It may turn detrimental to all.

A serious but nonetheless rather wide-spread mistake is assigning the length of piles based on their bearing capacities.

One often hears statements like: “We have tested a pile, haven’t we? Its bearing capacity is good, and its settlement during testing is only 2 cm!”

Dear Sirs! You maintained the load on the pile only for several hours, and the house will have to stand on piles for one hundred years! There is a difference, you see. Such safely tested piles developed settlements of more than 30 cm for a series of houses on Vasilevsky Island in St. Petersburg, whereas it had been supposed the settlement would only reach 4 cm, as the tests had shown. In first two years settlements were still insignificant. New residents moved in. And in 20 years dangerous cracks appeared and the lifts jammed.

Consequences of trying to save on pile lengths: a tilting house, cracking panels, settlements of about 0,5 m!



Settlement, mm

A bright example of a wrong choice of pile length is one of the houses built in St. Petersburg's Lakhta. Hapless designers decided to "rationalize" the project: from 21 m they truncated the piles down to 7 m! Pile toes got embedded in weak soils. The house settled by half a meter and developed a tilt of about 1 %. A little more and such tilt of an apartment floor would have been considered a steep descent on a highway.

It is not worth it trying to use half-length and half-price piles. It may turn detrimental to all.

How to arrange the underground space?

In greenfield conditions constructing a shallow underground structure sometimes appears possible even in an open excavation pit with slopes. If you need to go deeper than the underground water your excavation must be encircled by a waterproof cut-off screen. Good quality sheet piles hold water, whereas the sheet piles themselves are kept by the slopes.

Prior to constructing the foundation pit one should have thought about the existing buildings. Even about such unsophisticated ones.

If there is no place for slopes, it is necessary to make a cofferdam calculated based on reasons of stability. In greenfield conditions its horizontal displacements are not so important for us.

As soon as an existing building "appears" in the vicinity the situation becomes complicated. The main issue becomes calculation of the cofferdam movements, rather than calculation of its stability. Unfortunately, not all designers understand this and often mistakes are made. It badly affects the existing buildings and the client may have to face additional costs.

If in greenfield conditions no one will notice a movement of sheet piles of 10 cm, whereas a 3...4 cm settlement of an existing building will lead, at least, to a high-tension outburst of public passions. Here it is possible to notice an important link: settlements of an existing building in adjacency are approximately equal to horizontal displacements of the sheet piles.

If additional settlements of an adjoining building should not exceed 2 cm then a displacement of the cofferdam should likewise be not more than 2...3 cm. It means that you should both construct shoring of the cofferdam and make the cofferdam itself rigid. It is certainly not cheap but in this situation there is nothing else you can do. Such is the reality of geotechnical practice in any region of the world.

Settlements of an existing building in adjacency are approximately equal to horizontal displacements of the sheet piles.

For a cofferdam, as a rule, the so-called diaphragm wall or sheet piles are used. Without going into much technical detail, the following recommendations can be provided.

If your foundation pit is located 0,5 m above the footings of existing buildings and above the level of ground water, it may be possible to construct your foundation without any cofferdam.

If you need to excavate down to the footings of existing foundations a sheet pile wall may as well suffice.

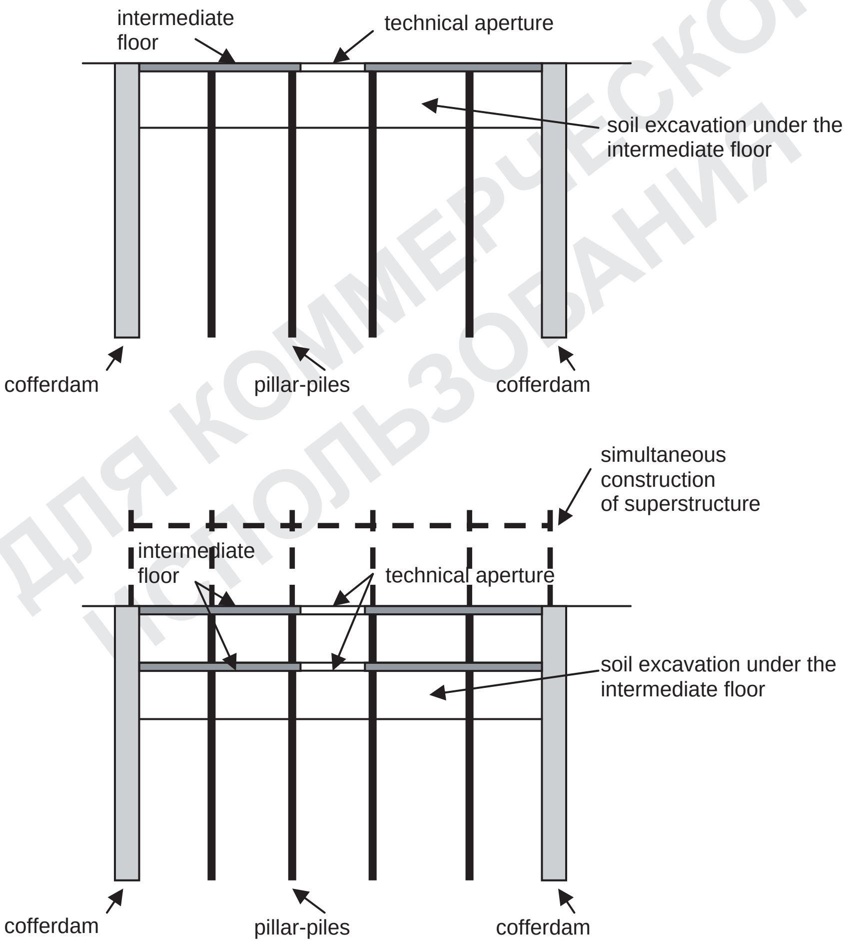
But for deeper foundation pits in the geological conditions of St. Petersburg just making a cofferdam (from sheet piles or diaphragm walls) will not be enough anymore. A foundation pit with depth of more than 3…4 m requires a shoring system preventing the cofferdam from horizontal movements.

Usage of anchors holding your cofferdam in place is not possible in our weak soils, as they do not give an opportunity to gain the necessary bearing capacity without encroaching on the neighbouring territory under the existing buildings where the use of anchors is unacceptable and extremely dangerous.

Examples of cofferdam construction using sheet piles (left) and the diaphragm wall (right).

What remains is the option of the shoring system which can be constructed using structural steel or reinforced concrete. Those should be used to prop the cofferdam carefully on several levels in the process of foundation pit excavation.

At this point we should talk specifically about the method known as “top-down” construction. In this method the function of shoring elements is performed by intermediate underground floor slabs with big or small apertures. In conditions of weak St. Petersburg soils excavation of a multi-storey underground volume using the “top-down” method is very complicated. In the subterranean world, as in Dante’s “Inferno”, it is dark, wet and dirty. Machinery sinks, and so do your feet. Therefore the method should be applied only when there is no alternative, namely, in immediate adjacency to existing buildings. To sugar the pill of this method one should remember the fact that it allows to simultaneously build the superstructure. It is possible to see in the west or in Moscow a 40-th floor of a skyscraper constructed at the same time with the minus four level of an underground parking facility. To carry out such synchronous construction upwards and downwards the designer should be well aware of all the nuances of this method.



The "top-down" construction method.

Choosing the design of your cofferdam it is necessary to bear in mind that the structure is capable of caving in under the pressure of the surrounding soil. It is especially dangerous in locations where we have not yet had the time to install the next level of shoring props. Therefore, the cofferdam itself needs to possess sufficient rigidity. The maximum rigidity is achieved with the diaphragm wall, especially when using additional buttresses. The wall itself can be up to one and a half meter thick, and a buttress extending by 3 m can transform a meter-thick diaphragm wall into an equivalent one of 2,5 meter thick. Such rigidity will never be achieved by a sheet pile wall. The most sophisticated sheet piles protection will hardly ever be more rigid, than a diaphragm wall with thickness of 800 mm.

One more way of cofferdam construction is known, but it has had very bad history in St. Petersburg. Here we are talking about the secant pile wall method. We shall talk about disadvantages of this method, which had caused destruction of several buildings in St. Petersburg, in Chapter 15, devoted to geotechnologies. Now it will be enough to point out that rigidity of such a wall is several times lower than that of a monolith diaphragm wall of the same thickness. Additionally, a secant pile wall cannot be entirely water-resistant.

Chapter 7, explaining the design

We have approached one of the most important moments for the manager (or the client) of a construction project – the choice of the designer. And it is at this point that we should reveal one terrible secret. It appears that no educational institution in Russia "releases" complete and accomplished designers. The reason for this is very simple and is veiled by the history of Russian higher education system. A graduate of a civil engineering college always had three roads open for him or her: construction industry, a design organization or a scientific institution (*i.e.* a research institute or a university department). In the times of the Soviet Union when science was respected, the last way was always the most prestigious. Choosing that path, people enlisted in postgraduate study programmes and defended PhD theses. The Soviet measure of a 400 roubles salary paid to an assistant professor (let alone professorial positions of high standing) would have been enviable enough for any construction superintendent, foreman or designer.

In the meantime, another graduate who upon graduation had been enlisted into a design bureau was gathering experience in practical design, supervised by aces of the trade. In about seven years, receiving about 250 roubles a month, he or she would start looking back at the *alma mater* thinking whether it might not be altogether worthless to return and teach, sharing experience of real design.

And at that point he or she would find out that, in view of no PhD degree being in place, the chances to receive any salary above that of a department caretaker (150 roubles) were very slim indeed. Therefore amongst university lecturers the real designer was the rarest of exceptions.

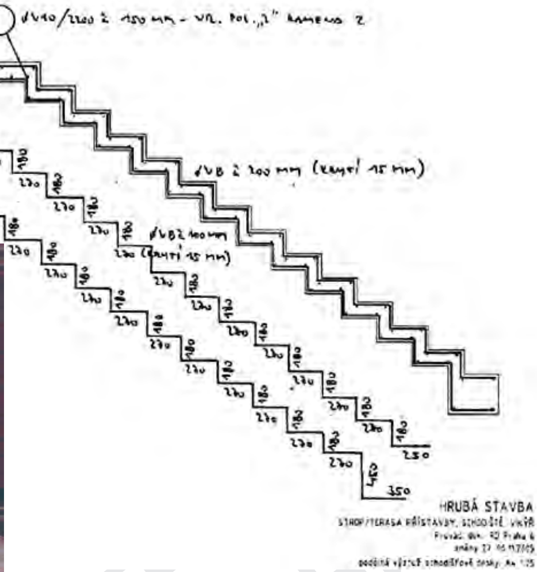
> You are advised to look for a design bureau which has retained its schooling, not just its sign on the door.

Once, in long-forgotten pre-perestroika times, in leading civil engineering high schools of the country in order to get a postgrad a two-year-old experience of real design work was necessary. Thereafter this requirement was somehow forgotten.

After perestroika the high school was abandoned by almost everyone for whom teaching was not a calling, not a real vocation.

Thus, these days in the high school there are almost no lecturers who know and understand design, and consequently design is barely taught at all. Hence, if you are in search of a designer, contacting a college or a university would be quite meaningless. If on people's business cards you see abbreviations such as PhD or PhD CEng, they are indications that, probably, you have got a worthy and clever person standing before you and a very good researcher at that, but most likely poorly connected with real practice of construction design. And remember to be especially suspicious of those holding the rank of an academician.

Today all kinds of academies have bred out of any reasonable proportion; their members do not even always have the rank of a Candidate of Sciences (roughly equivalent to German unhabilitated PhD); some of them have gone so far as not to possess even a higher education certificate. Such "academicians" pose a real threat when engaged in design, solving safety issues for hundreds of townspeople at their own obfuscated discretion.



Uneducated design and its result.

Designers were educated only at big Design Institutes. The fact that those institutes have largely perished today is a tragedy for domestic design business. Together with them Russia lost its school of building design along with the time-honoured methodology and the system of assigning responsibility.

Today in St. Petersburg there are very few design organizations which have kept their schooling, and not just their sign on the door. Fortunately, "Georeconstruction" Institute is among their kind, having rescued well-formed teams of designers from leading St. Petersburg institutes including Promstrojproject, PI No 1, Fundamentproject, and VNIIGS (the last mentioned institute was the leading USSR authority on underground construction but disappeared during perestroika). It was only owing to these people that the present writers (who also have titles of PhD and PhD CEng, or, in other words, good scientific affiliations) became designers. Many years of teamwork were necessary to achieve this status. It was the hard way that the authors came to experience the difference between the approach of a scientist and a real designer. For the scientist the negative result is indeed a result, and not a mistake. The designer, however, has no right to obtain such a mistaken result. He is the person who appends his signature to drawings and has a life-long criminal liability for correctness of the solutions therein expressed. He is the parent of the child, whose name is "the design".

When the awning above the underground station "Ploshchad Mira" collapsed, a 90-year old grandmother of one of our employees had a visit from a detective inspector who came to conduct criminal investigation, as it was her signature that he had seen on the ill-fated awning's drawings.

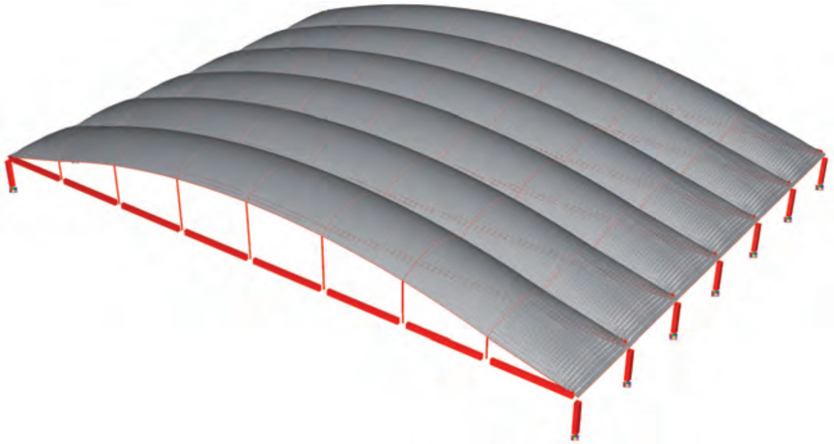
Now you understand why the True Designer is sometimes not very flexible with your demands: the Investor is responsible with his purse, whereas the Designer – with his head.

> **A professional designer will have a safety coefficient only where it is really necessary, whereas an ignoramus will not be saved even with a significant overuse of materials.**

The True Designer will not be willing to experiment, neither will he be prepared to try out presently fashionable, but untested solutions, materials and technologies. The True Designer will never be goaded into violating the codes. It does not mean that he is a stubborn reactionary. Creations of our employees, reinforced concrete flights 100 m long and only 10 cm thick, still excite the onlooker with the elegance of their technical solutions.

It is not necessary to think that the most reliable design solution is necessarily also the most expensive. A professional designer will have a safety coefficient only where it is really necessary, whereas an ignoramus will not be saved even with a significant overuse of materials.

In the early 1990-s our highly experienced employees specialized on perfecting other people's design solutions. On one project they managed "to relieve" the design of 6 railway car-load of rebar from those places where it was absolutely unnecessary and to add one carload to the place where reinforcement was insufficient.



The scheme of 96 m-long barrel-shaped awning for the main building of bus station No 6. The author of this solution A.V. Shapiro, who for many years was the head of Georeconstruction's Structural Design Department, received the State Prize for this work.

Often a high school scientist appears to be much braver than a designer. He boldly signs conclusions and reports, generously dispenses oral counsel, advocates "new technologies". Before following his advice we recommend that you find out what kind of responsibility the man bears. More often than not he bears no responsibility. All responsibility and liability lie with the designer who signs drawings.

However, the designer who has not been through a true school at times does not realize the full extent of his responsibility. He may be gullible enough to believe another's irresponsible advice, thereby cooking his own goose and getting the investor into financial trouble. Unfortunately, there have been multiple examples when tenants who bought properties in unreliable buildings suffered severely.

Very often design solutions presented as "bold" and "economical" are nothing more than robbing projects of their reliability.

Allow us to relate to you a real story. One of our employees, a professional designer with long-term experience, decided to purchase an apartment in a house of which one superstructure level had already been constructed. Having glanced at the design drawings she expressed her surprise: "But you really should have piles here, not the slab!"

And it so happened that the project had just been submitted to the State Expert Board who came to the same conclusion. Construction was suspended, the investor began to look for a way to introduce piles under the existing slab. It was more complicated and expensive than to demolish the structures already completed and start again. But pulling the building down was unthinkable – it would have scared away the shareholders – the future residents. In a year our colleague again inquired as to the matter and received a grumpy answer: "You did say piles were necessary, didn't you? So, we are putting them in, aren't we?"

It was found out later that not only did the design lack piles, it also had the working reinforcement aligned in the wrong direction (along, rather than across, the walls). The originator of the design had been a young wet-behind-the-ears graduate, who by misfortune, happened to have been appointed Senior Designer in an architectural workshop. One really could not help but feel pity towards him – he had never been taught how to design things. But one also had to pity the unlucky shareholders.

Fortunately, in the God Protected Russia it is not each incorrectly designed house that falls. For example, a house in Dvinskaya Street endured for 30 years before collapsing, despite the bad mistakes made during site investigation, design, construction and maintenance (see Chapter 22).

Very often design solutions presented as "bold" and "economical" in essence are nothing more than depriving projects of their reliability. We are not talking about saved money here; what we are talking about is postponed risks, for which someone will have to pay. It is quite possible that we shall yet have to reap the fruit of the building boom which happened during the time when True Designers were few and far between.

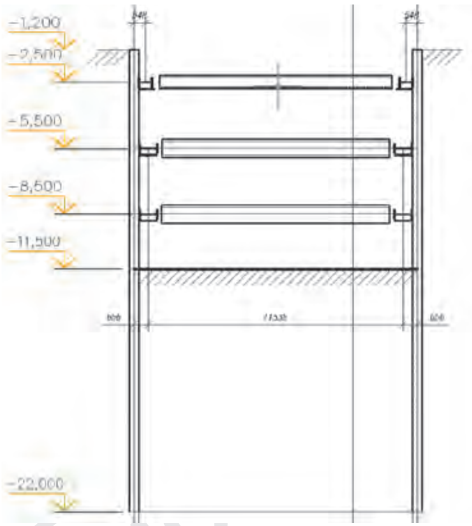
The house on Dvinskaya Street before collapse.

For complex projects, such as underground sections of buildings in congested city conditions, a good designer will necessarily envisage a procedure commonly referred to in the West as "observational method", or interactive design. This method allows to resolve challenges for which there is insufficient construction experience and for which technical solutions are not yet properly honed. The method excludes unreasonable and unnecessary safety coefficients, simultaneously ruling out dangerous deficiencies. Interactive design is organized according to the following pattern: *initial design – test site – updated initial design.* The initial design contains various solution scenarios – from optimistic to pessimistic. On the test site, which is a characteristic fragment of the future structure, deformations, and loads in superstructure and subsoil are measured.

Complex projects require interactive design structured as follows: initial design – test site – updated initial design.

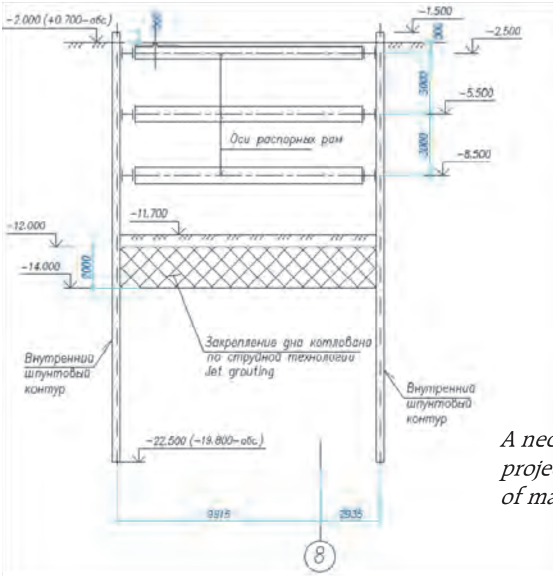
It is determined which of the scenarios meets the reality. After that, the initial design is updated if necessary. This approach achieves maximum economy at optimum reliability. It is often possible to arrange a test site so that it later becomes a part of the future building. In this case additional expenses are required only for installation of additional measuring instrumentation. It was this approach that we implemented for designing the underground sections of the Second House of Mariinsky Theatre in St. Petersburg. Results obtained from the *in situ* measurements on the test pit we had arranged became the basis for the whole discipline of underground structure design in St. Petersburg.

1. Initial Design



2. Test Pit

3. Updated initial design



A necessary sequence of complex project design ruling out overuse of materials.

Here is some advice how to find out if there is true design expertise in a design organization.

First thing is not to trust signboards of even the oldest institutions. For many of them their glory is only the stuff of legend. It is far better to trust personal names.

If you do not know the designers, pay attention to their age. A true design school has all generations more or less evenly represented: from 25 to 70-year olds. If key solutions about the project are taken by a person younger than 30, regard them with suspicion. Obviously, such a person lacks the company of senior colleagues and there is no one he or she could learn from.

You can only prop yourself against something that resists. Such is the law taught in Strength of Materials. It works both in science, and in business.

Pay attention to how the designer requests initial data from you. The True Designer regards the matter with utmost importance. The more exacting the designer is when accepting initial data, the more uncomplicated he or she will be to work with. The ideal situation is when site investigation order is drafted by the person who will later use the data in the design.

There is one more attribute the True Designer exhibits. He thinks broadly, is ready to realize most courageous architectural ideas, but will be most unflinching in elimination of architectural mistakes. When issues of durability, reliability and stability of structures are at stake, the True Designer becomes most pigheadedly stubborn. He or she will never be open to compromise to the detriment of public safety. Dear Manager, it is possible to rely only on such Designer! You can only prop yourself against something that resists. Otherwise you will sink, as if into a bog. Such is the law taught in the subject known as Theory of Strength of Materials. It works both in science, and in business. To rely on cotton-wool subordinates is the end of all business.

Part Two

A Taste of Calculations

This part is strictly unessential. Calculations, (we might as well have used terms like "computation" or "analysis"), especially those connected with soils, are quite a specific dish which necessarily will not suit all tastes. Therefore those who do not wish to go deeply into particulars of calculations, can delegate appreciation of calculations quality to experts and carry on with their further travel through geotechnical engineering skipping to Part Three of this book.

Chapter 8, explaining expert evaluation of calculations and computer programs

Probably, in each trade experts like to surprise the uninitiated with instant evaluations of complex problems, inaccessible to the average punter. Some doctors can diagnose an illness having merely glanced at a patient. Engineers too may like to impress others with an instant evaluation of a problem. Certainly, experience plays a huge part here. But experience alone is not enough. One's "inner voice" might as well be wrong. Engineering problems are very diverse, and it is seldom that one can encounter absolutely identical problems. Intuition works only in the field of qualitative assessment. "To feel" a numerical value of a calculative parameter is a task inaccessible for intuition. Skilled engineering designers always give a *quantitative* assessment of a situation, an approximate one, yes, but always quantitative. (By the way you should be wary of engineers who, explaining a technical solution to you, will refer only to experience and intuition, instead of calculations. It may be the case that such engineers simply do not know how to calculate).

> **Do not trust intuition alone. One's "inner voice" might as well be wrong. Calculations are indispensable.**

It is very curious to observe dialogues between young engineers and skilled designers. The young specialist, having received results of calculation from a computer program after a week of persistent labour, shows them to the Senior Designer. The latter, in about 30 seconds, by means of a simple accounting calculator, utters the verdict whether the calculations are right or wrong. What is the nitty-gritty here, you might ask? Is it really so that the an expert is capable of replacing complex computer programs with his intuition?

Certainly not. The art of the engineer is, in many respects, the skill to describe complex phenomena with approximated but nonetheless simple and clear numerical schemes. This is the art of approximate assessment of loads in structures. This present chapter is devoted to methods and devices of such approximate calculations.

The art of the engineer is the skill to describe complex phenomena with simple and clear numerical schemes.

Approximate assessment is not taught in the high school, nor will you be able to find it in literature. The matter is that such calculations are, strictly speaking, *not quite correct*. They are, indeed, reasonably rough and approximate, yet still giving you a sense of the true order of values. Because of their roughness, these calculations are not explained in any textbook. But in reality, people engaged in long and difficult *correct* calculations sometimes do not see the wood for the trees. And it is in this case that the approximate assessment is very useful to identify errors.

What use is certified calculation software?

Recently engineers have been ever so more often using computer programs for complex calculations. Clever people understand: Whatsoever a man inputeth that also shall he output. The program is only a tool, and, as it is known, one could even knock nails in with a microscope, if push comes to shove. Unfortunately, wide distribution of programs has spawned a generation of hapless experts, who, having learned to press buttons, think that they have mastered the art of calculation. Very often they pompously retort: "I've got a complex problem, and hi-end certified software to solve it! What are you trying to prove here with your calculator?!". This "computer" epidemic hit even more people abroad than in Russia, because the majority of population there began to use computers earlier.

Once we had to prove to our British colleagues that a superstructure of a four-level parking facility cannot load its subsoil with a weight comparable to that of a sixteen storied building. The answer sounded thus: “Well, we don’t know, really... We used a computer program...” It is possible to calculate on an abacus, or on a computer – how you calculate is not likely to change the law of universal gravitation!

No thrice-certified program can guarantee correctness of your calculations. Developers of programs’ ***do not bear any responsibility for calculation results.*** But if an engineer is mistaken it is his mistake and his responsibility. The point of programs *certification* is their conformity to separate clauses of codes, standards and norms. And it needs to be borne in mind that *certification* and *verification* are, indeed, different concepts. Certification does not imply *verification*, *i.e.* more or less detailed check of calculation results a program is able to yield.

No thrice-certified program can guarantee correctness of calculations.

Rules of building mechanics are not stated in codes. Therefore a certification does not concern correctness of calculation of loads in structural elements. What it concerns is only rebar, cross-sections, *etc* that the program selects. Therefore a program which, say, only computes loads in beams (and does not select rebar) is impossible to certify. The sad conclusion from here is that a certificate is, in general, a Mickey Mouse paper, needed entirely to assist sales of software. Sometimes an uncertified long division sum on a scrap of paper can be more correct.

It does not mean at all, of course that everything should be calculated on scraps of paper using long division and computer programs should be done away with. It is just that we, being developers of one of widely known soil-structure calculation programs (*FEM models*), know well that reckless trust to results of computer calculation is extremely dangerous. A good reaction to any calculation results is that of healthy mistrust. It is not until several series of calculations (including also analytical ones) have been performed that a conclusion can be drawn about validity of their results.

A good reaction to any calculation result is that of healthy mistrust.

What is the complexity of writing simple programs to calculate according to SNiP formulas?

It would seem that creation of finite element programs is essentially more complex than writing simple calculation software according to formulas expressed in normative documents, such as SNiP. Really, finite element programs feature much more refined mathematics than simple arithmetics underlying engineering calculation formulas. Nevertheless "glitches" in simple programs are encountered even more often than in hi-end finite element complexes. What is the matter and why is this so?

Those who have ever dealt seriously with programming know that the biggest amount of time of the programmer is spent not on writing the program but on its fine-tuning and debugging. At times it is easier to write a difficult complex of programs for finite element calculations, than to consider all notes and conditions to some table from the local codes. As construction codes often have empirical character, algorithms of calculations according to their formulas possess plenty of conditions. Programmers know that debugging programs requires checking how each branch of the algorithm works. In order to track all bends in the branches of the algorithm tree it is necessary to create a huge quantity of test examples. As a result, a full check of all opportunities of the program becomes rather labour-consuming, and sometimes even practically impossible. We shall here give a simple example explaining complexity of fine-tuning such programs. Let us assume that a program incorporates a table of empirical factors with the size of, say, 10 by 10 cells. In one of the cells the programmer made a mistake, writing in a wrong value. As a result 99 tests will be absolutely correct, and only 1 will produce complete nonsense. “Fishing out” such a mistake in the program is very difficult.

From here a simple conclusion may be drawn: healthy mistrust should be present also when dealing with results of simple calculation programs using formulas of construction codes. Certainly, there exists good and absolutely faultless software. However, the principle of “a black box” (when it is not possible to see what happens to our initial data during computation) is quite dangerous to engineering calculations. The engineer, performing a calculation, should still realize what he or she is doing in the process. Certainly, we do not suggest to throw away computers and to calculate everything by means of slide rules. Calculations in convenient mathematical packages such as, for example, MathCAD, appear more promising, as the entire text (and most importantly the essence) of the calculation is visible, and the computer simply assists in performing arithmetic operations.

How to check computation results?

The main problem of computer calculations using the method of finite elements is complexity of their verification. A real verification of a calculation can only be done by the analyst himself or by an expert to whom all files have been given. It is possible of course to check a calculation, having repeated it completely from the very beginning (by the way, this is a mandatory procedure abroad).

As an output of calculations the customer is given a set of beautiful pictures which he is encouraged to believe. Unfortunately, irresistible attraction of computer graphics is not a token of results' truthfulness. How then is it possible to evaluate correctness of calculations?

There is a recommendation of the State Expert Board to check calculations using two different programs. There is little sense in such a check. If two programs solve an identical problem in similar ways, an error incorporated in calculation will lead to identical (but equally erroneous) results. On the other hand, specific features of mathematical description of some elements in different programs can generate divergence in the so-called "special" points (where exact solution by means of finite element method cannot be obtained). An expert badly versed in matters of numerical calculations will be led to think that results are essentially different, and, hence, one of the calculations is wrong.

Actually it is necessary to compare calculations performed according to different numerical schemes, with different breakdowns into finite elements, using different types of elements, done according to various approaches, including simple analytical ones. All this might as well be done within one program. The requirement to calculate by means of different programs allows to boost software sales (indeed, this requirement appeared mainly with exactly that purpose in mind).

It is possible to evaluate the level of an analyst asking him to explain his calculation results.

It is possible to evaluate the level of an analyst asking him to explain his calculation results. A true analyst considers his work to be done only when there is not a single unclear effect in his calculation results. If in response to your question you hear things like: "This was the output", "That's the way my software did it" it is not the person you need to be talking to. For revealing all the reasons of all the phenomena, as a rule, instead of one problem it is necessary to solve ten, applying various numerical models and changing numerical schemes.

Only when the entire series of differently solved problems leads to the same conclusion, it is acceptable to see the result as reliable. Of course, such work must be done by the analyst himself. For an independent inspection it is possible to apply simple devices and rules which will be discussed below.

> **Not until an entire series of differently solved problems has lead to the same conclusion, is it possible to see the result as reliable.**

How to check summation of concentrated loads in the subsoil?

Such problem arises very often. Certainly, for gathering of all loads it is necessary to calculate attentively the weight of all structures and to take temporary loads (people, furniture, snow, *etc*) from the applicable codes. However, to assess the order of values it is possible to use this simple stratagem. A cubic metre of a building weighs 0.5 tons (or 5 kN for those who prefer another scale of measurements). We checked this on different buildings, from standard five-storey residential blocks to a 400-m tall skyscraper – as a whole it works rather well. Differences, certainly, exist (sometimes a building happens to be a little lighter, sometimes a little heavier), but we will reiterate once again – it is quite enough for an assessment of the order of things. According to this rule a 5-storey building transfers to the subsoil average pressure approximately equal to: 5 storeys × 3 meters × 5 kN = 75 kPa, a 16-storeyed building: 16×3×5 = 240 kPa.

By means of simple arithmetics it is possible to check easily, for example, the quantity of piles under a building. For this purpose you need to take the area of a single storey, multiply it by the height of the building (thus obtaining its volume) and further multiply by 0.5 tons. We shall thus obtain the weight of a building in tons. Then it can be divided into the rated load on piles assumed in the design (which should be specified in the design of a piled foundation). As a result you obtain the necessary quantity of piles. The actual quantity of piles usually turns out a little bit bigger. The matter is that there always will be places where piles need to be installed as designers say, "for constructive reasons". The factor of constructive arrangement of piles, as a rule, is 1.1...1.2. If the number of piles is 1.5 times greater than necessary, most likely the designer has already reached a certain mutually satisfying agreement with the piling contractor. There can be of course completely disinterested incompetence also. On one project – a high-rise building – designers put in 4 times more piles than necessary. The contractors however were not happy at all as the piles were designed so densely that it was impossible to install them properly.

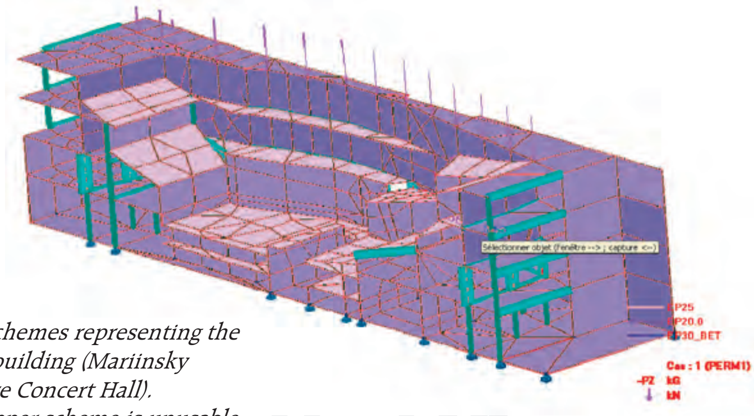
How to assess quality of calculations looking at numerical schemes?

Numerical schemes based on the finite element method, consist, as is well known, of tiny separate elements. For buildings they are, as a rule, walls and intermediate floor slabs, as well as rod elements for columns. Considering the scheme's outlook it is possible to draw some conclusions.

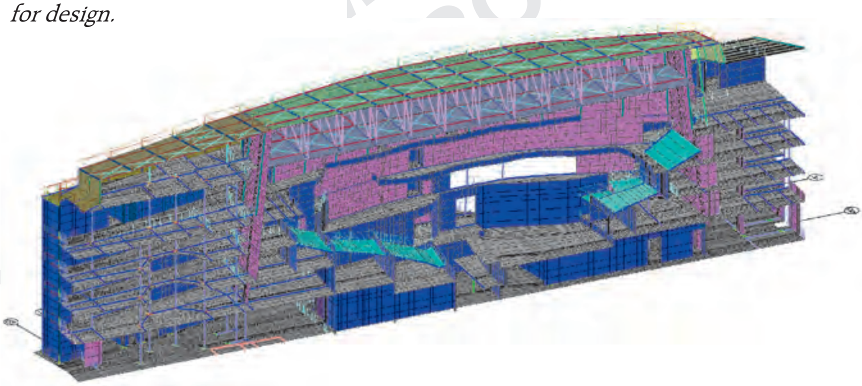
Each numerical scheme should correspond to the purpose of calculation being performed. The same numerical scheme can be correct for one purpose and wholly unsuitable for another.

If the purpose of calculation is general assessment of loads transferable onto subsoil and an overall estimation of how the building behaves and what are its general rigidity and settlement differential, for such a calculation we can use a rough scheme, say, with 1 final element for a storey height in the walls and 1...2 finite elements for a flight of intermediate floors. But such a scheme cannot be used to calculate loads in the structures!

If to you are shown pictures of reinforcement in intermediate floors broken down with less than 4 elements for one flight between bearing walls, you are unequivocally being taken to the cleaners. Finite elements in this case simply cannot properly represent a bend of the intermediate floor slab between the bearing walls, and the values of bending loads are extremely incorrect. In this case, may God have mercy and enlighten a skilled designer to simply ignore such calculation results and recalculate the slab himself.



Two schemes representing the same building (Mariinsky Theatre Concert Hall). The upper scheme is unusable for design.



Unfortunately, there are cases of blind trust to results of calculation. One foundation slab was designed by a hapless "end user" exactly according to results of machine calculation – for each square meter of the slab the drawings had a reinforcement cage with rebar positioned exactly according to the computer output. The trouble was that numerical model was not only crude but also generally wrong, and the working rebar had to be placed on the other side of the slab.

In an assessment of calculation results obtained according to the finite element method it is necessary to remember that this method is numerical, which means that its accuracy is always limited. The task of a competent analyst is not only to know the boundaries in which the result is correct, but also to know *what* result is correct in *what* boundaries. For example, a rough grid is unsuitable for the choice of rebar.

> **The task of a competent analyst is to know what result is correct in what boundaries.**

When calculating a building together with its subsoil it is necessary to pay attention to the size of the numerical scheme used for the subsoil. If the scheme models soil (in flat or spatial setting) the size of the soil bulk needs to be quite big. The numerical scheme should be cut off at a distance not less than 2…3 sizes of the compressed bulk in all directions from the edge of the building. It is especially important to pay attention to the scheme size when calculating cofferdams. Recently a few schemes were brought to us for examination in which the cofferdam was practically "tied" to the edge of the finite element grid (*i.e.* outside the pit a soil bulk of, say, 10 meters was drawn at the cofferdam depth of 30 m). In this case the cofferdam "holds on" to the edges of the scheme and does not fail, which infinitely pleases the Mickey Mouse analyst. However on a building site, no matter how strongly you desire, the illusory edge of the numerical scheme cannot contribute anything to subsoil strength.

How to quickly assess sufficiency of reinforcement in a reinforced concrete structural element?

As is well known, in reinforced concrete the concrete is responsible for compression, and the rebar – for tension. Therefore, when a concrete element bends, the ultimate moment creates the ultimate load in the stretched rebar, multiplied by distance to the center of the compressed concrete zone. Students are taught to make such calculations correctly, with definition of the sizes of this compressed zone. However, it is possible to calculate approximately, but in a much easier way, having measured the distance to the center of the compressed zone "by rule of thumb". Then the entire check will consist in multiplication of three numbers: square area of rebar, its working resistance (365000 kPa or 36500 ton/m^2) and that approximate distance. The square area of rebar is convenient to assume from tables or to calculate according to the elementary "school" formula of the area of a circle (in the further calculation it is only necessary not to forget to translate measurement units from centimetres or millimetres into meters).

For example, we have a slab with thickness 800 mm, reinforced with rebar type A III, diameter 20 mm, spaced at 200 mm. One meter of slab takes five rebar. Using a table (or the formula for the area of a circle) we calculate the area of 5 rebar rods as 15.7 cm^2 or 0.00157 m^2. Now let us take the approximate distance from the rebar to the center of the compressed zone as 0.75 m (having excluded from calculation the protective concrete layers). Having multiplied the square area of rebar by its working resistance and by that approximate distance, we obtain:

$$0.75 \times 365000 \times 0.00157 = 429.8 \text{ kNm}$$

The exact value of the ultimate moment (if we calculate thoroughly) is 427 kNm, so our solution appears quite good for an approximate assessment.

Chapter 9, explaining calculations of buildings' settlements

Talking about assessment of calculation results for settlement of buildings turns out to be much lengthier, because calculation of settlements is a very complicated problem. Therefore we shall break the issue into a series of smaller ones and shall begin with calculating settlements according to techniques expressed in normative documents.

Normative methods and their correctness

Even more or less recently one could find in normative documents some settlement calculation methods for buildings:

- the method of layer-by-layer summation SNiP 2.02.01-83*;
- the method of linearly-deformable layer SNiP 2.02.01-83*;
- modified method of layer-by-layer summation SP 50-101-2004;
- the method of settlement calculation for piled foundations SP 50-102-2003;
- the method of settlement calculation for pile-raft foundations SP 50-102-2003.

The first three methods are based on the same mathematics, *i.e.* the solution of Boussinesq problem applied to the border of unlimited elastic half-space. It would seem that the same mathematics should lead to affine solutions. But far from it…

All normative settlement calculation methods artificially limit thickness of compressible stratum.

The matter is that it is impossible to directly calculate settlement according to the theory of elasticity. And not only because soil works non-linearly. According to the theory of elasticity, deformation under a foundation rather slowly fades with depth. If you sum the entire stress epure down to the Earth's core, you receive an absolutely unreal value. Therefore all existing methods restrict the depth of compressible stratum one way or another. In the method of layer-by-layer summation the compressible stratum is limited to the depth on which additional stress from the foundation does not exceed 20 % of the stress caused by proper weight of overlying soil strata. In the method of linearly-deformable layer depth of compressible stratum depends on stress rather poorly, and is determined basically by width of the foundation. In SP 50-101-2004 a new modification of the layer-by-layer summation method was introduced, having eliminated the linearly-deformable layer method, changing also the criterion of restricting the depth of compressible stratum for broad foundations (50 % instead of 20 % of stress from the proper weight). It goes without saying that this change cardinally influences settlement values.

The methods for calculating settlements ***of piled foundations*** are based on the following approach: using a semi-empirical formula, settlement of a single pile is calculated, following which, using tables, the transition is made to settlement of pile group. Without getting too far into the scientific jungle, it is necessary to mention that such approach is at least debatable. As is well known, a single pile works, mainly, utilizing skin friction; however, when we consider settlements of a pile group, the common approach is that of "theoretical foundation", uniting all piles at their toe level. Between one pile and the theoretical foundation within the limits of the entire pile field there exists a qualitative, and not just a quantitative, difference. The main feature of such an approach to calculating settlement is that over the transition from settlement of one pile to settlement of a pile group it is apparently possible to arrive at settlements of any magnitude. In SP 50-102-2003 after a long description of the method to calculate settlement of the pile-raft foundation, as though in jest, it is later prescribed to check the obtained settlement value by recalculation using the old method of layer-by-layer summation.

So, we have several methods with absolutely different empirical approaches to settlement calculation. It is not surprising then that using these methods it is impossible to arrive at identical settlement values, and the engineer remains in perplexity with a line of numbers significantly different from each other. Which method to trust? The answer to this question can be given only by a skilled geotechnical engineer. It may seem that all this is invented specially to rid the civil engineer of any desire to be engaged in calculation of settlements, *i.e.* to provide the geotechnical engineer with work. However, below we shall try to unravel this confusing story.

Comparison of settlement calculations with real *in situ* monitoring

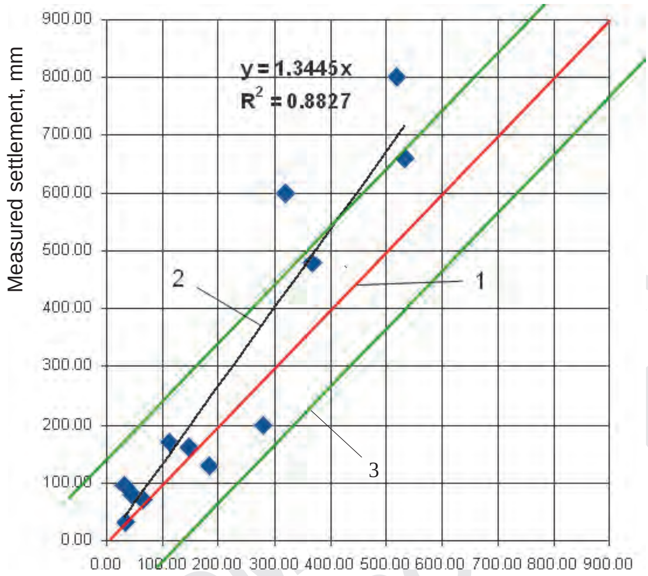
As is known, criterion of truth is practice. It would seem reasonable to think that millions of buildings have been constructed in the world and geodetic instruments have been in existence for rather a long time also. Over many years an enormous pool of data on buildings' settlements would have been formed and, based on the monitoring results, it should be possible to work out an elegant and accurate theory of settlement calculation. Alas, nothing of the kind has ever been done.

Beginning our work in Technical Committee 207 "Soil-Structure Interaction and Retaining Walls" of ISSMGE (International Society for Soil Mechanics and Geotechnical Engineering) we were quite naive to believe that clever people abroad must have accumulated loads of well documented settlement monitoring data. However, our western colleagues reasonably pointed out to us that over periods of long-term measurements settlements can turn out to be in excess of what had been expected. Who then will be responsible for them (also financially)? Therefore people involved in construction prefer not to monitor settlements of buildings once construction is completed. A similar situation is observed also in Russia. As a rule, settlement monitoring is conducted during construction of a building when the larger portion of settlement has not yet had enough time to accumulate. Long-term settlement observations are usually organized when there had been some problems during construction. There interests of many participants to the process can be infringed and consequently measurement data are held close to their chests. Consequently, there is a deficit of monitoring results as far as development of scientific theories is concerned. Because for a development of a settlement calculation theory, information on buildings' structural layout, site geology and monitoring data as such must be collected and grouped together. This challenge is rather serious.

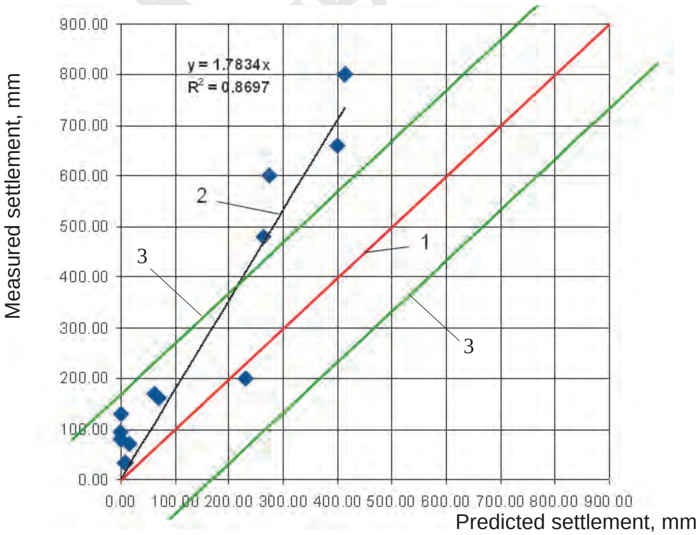
For St. Petersburg we managed to create a database for long-term settlement monitoring consisting of 15 buildings. For such a big city this number is awfully insignificant. However, today this database is one of the most representative in the world. By hook or by crook we have been trying to replenish it. For the time being we shall reveal statistics on these 15 objects below.

Presently, it is necessary to mention one complexity which arises in comparison of calculations and monitoring data. Which settlement value should be considered as final? As a rule, real monitoring results look like a smoothly flattening out curve. This curve has one feature. Its psychological influence strongly depends on the scale in which it is plotted. If the horizontal scale is stretched it will seem that settlements are fading. If, on the contrary, one should stretch the vertical scale it will seem that no attenuation is presently visible. Therefore it is necessary to look at numbers. Attenuation of settlements is no more than 5 mm a year, which in St. Petersburg is not always possible to gain by waiting for it. On many projects which we added to our collection no settlement stabilization was ever observed. Therefore the concept of "final settlement" in these situations can be applied only theoretically (final settlement was taken as equal to the results of the latest monitoring round). True final settlement will obviously be greater, but who knows which it will be? In some sense the concept of final settlement is similar to indication of a broken clock which gives you the correct time twice a day. This way any final settlement (of a reasonable value, of course) will sometimes occur. It will be necessary only to timely announce coincidence of your calculations with observed reality! Therefore one needs to speak not about final settlement but about ongoing settlement, developing in time.

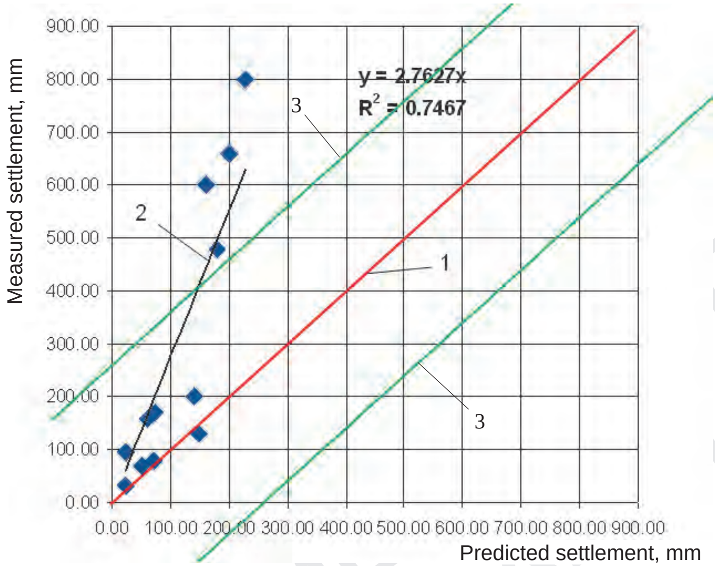
a)



b)



c)



Comparison of calculated and measured settlement values. a) – according to SNiP 2.02.01-83; b) – according to SP 50-101-2004; c) according to Egorov's Method.
1 – the line of ideal coincidence of measured and theoretical settlements;
2 – their linear approximation; 3 – mean-quadratic deviation.

The layer-by-layer summation method from SNiP works best: it is "only" 30% wrong.

So, now we shall compare the data of theoretical predictions according to different methods and the data of monitoring. Comparison is graphically convenient to be represented as follows: on the horizontal axis we shall place the predicted (theoretical) settlement values, and on the vertical – the measured ones. If they coincide, the points should lie along a straight line at a 45 degree angle. If points are densely grouped along the line it means the method distorts reality, but only a little. If points are significantly scattered – the method reflects the essence of the phenomenon to a very small degree. It is apparent from the above figures that for simple engineering calculation methods no good coincidence in general is observed. The best is the good old method of layer-by-layer summation – on average it is "only" 30 % wrong. Settlements according to other methods differ from monitoring several times. At this point geotechnical engineers ought to have strewn their heads with ash and left quietly through the door in order to find another job, in recognition of the fact that such accuracy is well comparable with reading teacups. Can this really be true that modern science can offer nothing to improve calculation accuracy? Below we shall try to find out the reasons for such amazingly low prediction quality.

The reason for low accuracy of engineering calculation methods

The reason for low accuracy of engineering calculation methods is actually very simple. Everyone who has ever studied strength of materials knows what even the most simple elastic material has at least two independent work characteristics (modulus of elasticity and Poisson ratio). But soil (according to geological reports) has only one, for some reason, the modulus of deformation. Where did the second characteristic go? It appears, it is not defined but assumed from tables. Having defined (barely, if at all) only one deformation characteristic of soil behaviour, we hope to obtain high accuracy. It is the same as if we wanted to devise a technique whereby to calculate people's weight by their height. For people of average constitution it would kind of work. But the reality is that there exist chubby fat shorties and thin basketball players! It is obvious that in order to improve the method of calculation, it would be quite necessary to describe the subject more precisely – to enter, at least, one more parameter, say, for example, a waist line measurement.

To research soil properties special devices were invented – triaxial cells – allowing to receive all necessary mechanical characteristics of soil, and to represent its behaviour dozens of various nonlinear models, containing different parameters, were created. But the trouble is that triaxial tests are seldom carried out, and the traditional site investigation procedures offer little else than one modulus of deformation. In this situation using complex nonlinear models is senseless – all other parameters of these models will have to be simply invented.

The reason for low accuracy of engineering settlement calculation methods is that soil behaviour cannot be described with just one parameter.

Thus the reason for low accuracy of engineering settlement calculation methods is that soil behaviour *cannot be described with just one parameter.* It is, in general, an obvious statement. But to correct the situation, it is necessary to reconstruct the entire system, starting from site investigation. It is necessary that the investor should commission more detailed site investigation with definition of special parameters for complex models of soil mechanics in triaxial tests (using triaxial cells). The program of such investigation should be prepared and written by a geotechnical engineer. It is necessary for the prospecting geologist to understand what kind of parameters he or she should define for the geotechnical engineer. Last but not least, it is necessary that the geotechnical engineer should be able to use complex nonlinear models and do his design work with these calculations in mind. This entire chain, unfortunately, does not work in the overwhelming majority of cases. The geologist and the geotechnical engineer (the analyst) often work in different organizations and do not wish to understand each other. As a result, even with the most detailed site investigation it is not always that adequate modelling of a building's behaviour can be done.

For one of the houses in the city centre our European colleagues designed piles with toe levels resting in firm Wendian clays (which, as we told in the chapter on geology, are the best kind of soil in St. Petersburg which it is possible to reach with piles). According to their calculations, the building of about 40 m in height would have the settlement of 9 cm. Our other foreign colleagues, having embedded a 400 m-tall building into the same firm clay, obtained settlement of about 2.5 cm. That is to say, settlement of a building 10 times taller was 3.5 times lower! It is obvious that someone was wrong somewhere (more precisely they were both wrong).

Let us look what results we could obtain if we tried to reconstruct the entire system of geotechnical calculations and to calculate settlements based on two deformative characteristics of soil behaviour using highly effective nonlinear models constructed according to data obtained from triaxial tests.

Some words about nonlinear models of soil behaviour

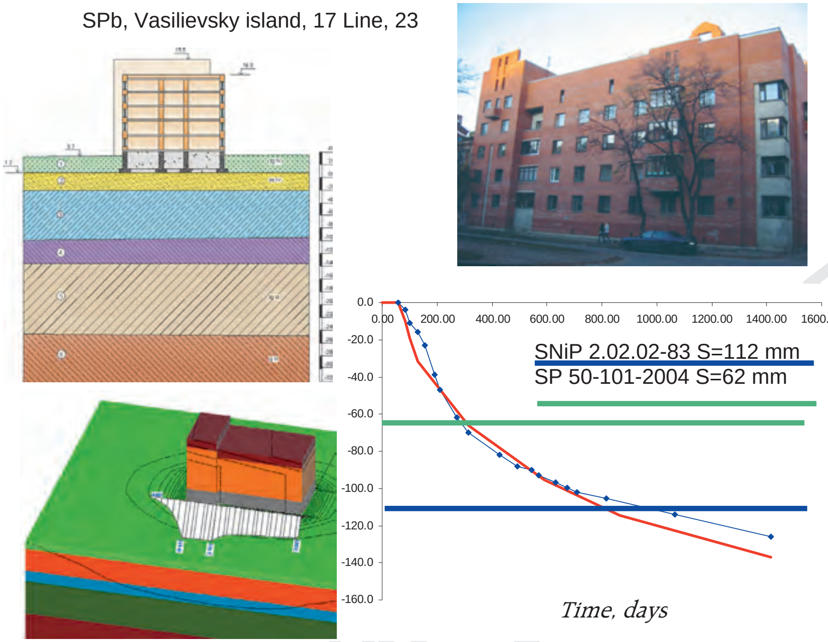
This subject, generally speaking, deserves volumes of literature to be written about it. But we shall try to say only the most important words, a few important remarks, not going deeply into this difficult matter.

At this point some analysts will say: so, what of those non-linear soil models? You just add soil strength characteristics and calculate with the same deformation modulus. Such model is usually referred to as "Coulomb-Mohr model" (although a more correct term for it would be the "ideally elastoplastic model with the limiting surface, described by Coulomb-Mohr criterion", but such name is cumbersome and long to pronounce). Unfortunately, it does not provide anything new by way of settlement calculation than the method of layer-by-layer summation. Indeed, we always limit pressure upon the subsoil and it is always far from the ultimate (when we get soil squeezed out from below the footing). And it means that nonlinearity will be rather weak and insignificant. Therefore, if using this model the analyst received something essentially new in comparison with the method of layer-by-layer summation, it means, that he has either bamboozled himself or is trying to bamboozle you.

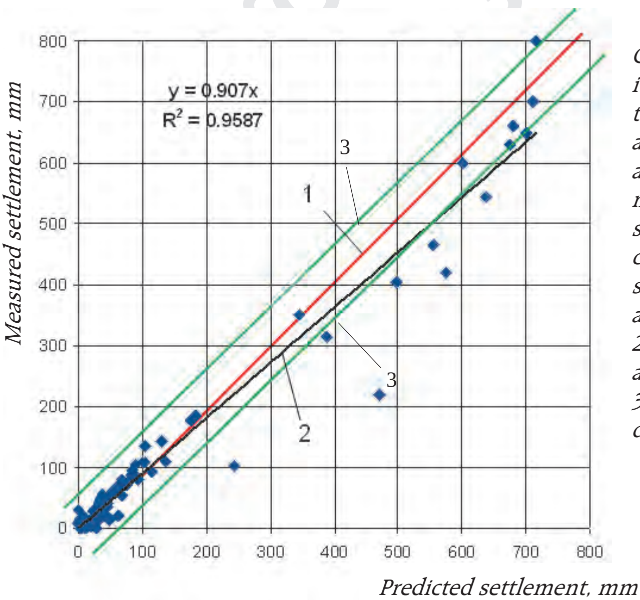
Except for the elementary Coulomb-Mohr model a huge array of much more complex and much more correct models of soil behaviour has been created, allowing to describe nonlinear compression and nonlinear shear based on results of triaxial tests. Examples of the most effective models is the Hardening Soil Model incorporated in PLAXIS software or the viscoplastic model of our program *FEM models.*

Application of nonlinear models in settlement calculations

From the database that we had collected, for the 15 houses, according to specially developed correlation dependencies it was possible to recreate detailed soil test data and to define parameters of a nonlinear model. Further on, it was just a matter of technical dexterity – to set numerical schemes for all buildings based on identical principle and to calculate settlements. For each of the buildings, based on the viscoplastic model incorporated in our *FEM models* software we calculated settlement propagation in time. The figure contains an example of settlement curves correlation as numerically predicted and as measured *in situ.* Such approach with modelling of settlement development allows one to get rid of the question about final settlement and to correlate calculations and measurements more correctly.



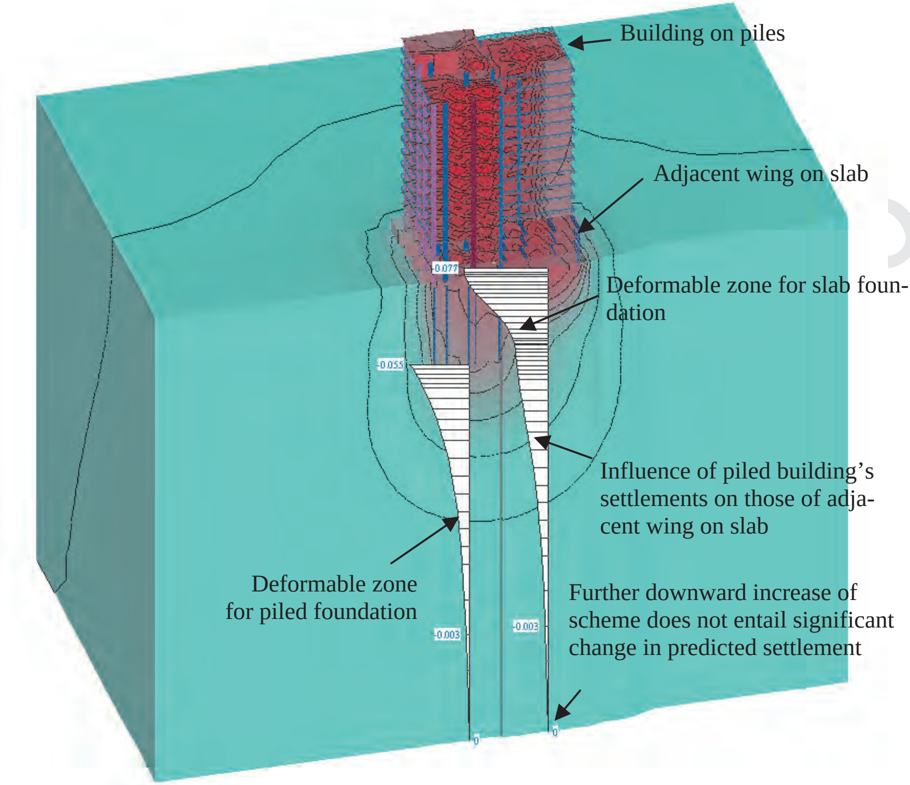
Monitoring data and settlement calculations by means of various methods for one of the houses in the database.



Comparison of numerical settlement predictions using a viscoplastic model and natural settlement monitoring data: 1 – straight line of ideal coincidence between settlement predictions and observations; 2 – their linear approximation; 3 – mean-quadratic deviation.

Let us attempt a statistical processing of the results. Same as before, we shall plot predicted settlement on the horizontal axis, and measured settlement on the vertical. The number of points has increased because comparison is being made for the different time periods – following one month, one year, and five years after the completion of construction, *etc.* We do come across unsuccessful correlations: in the figure it is possible to find points in which there is a 2 times difference between calculations and measurements. However, there are few of such points. The mean-quadratic deviation (which characterizes how much the chosen method of calculation reflects the considered phenomenon at all) is significantly reduced. And, most importantly, the scope of an average error is only 10 %. Another advantage of such calculations is that it has become possible finally to get rid of artificial restriction of compressible stratum which is inevitable when using more simple approaches. Here the nonlinear model limits the deformable zone in the subsoil by itself.

The viscoplastic model restricts the deformable zone in the subsoil by itself. Artificial restrictions are no longer required.



Advantages of applying nonlinear models to settlement calculation: the zone of deformations is limited automatically, it is possible to correctly model mutual influence of slab and pile foundations.

In principle, such result (after the gloomy pessimism of the simplified engineering methods) inspires certain confidence. It testifies to the fact that the chosen way, being based on more meticulous account of specific soil properties, is correct. On this way, certainly, much yet remains to be done. First and foremost, one needs to develop a habit to avail oneself of high quality site investigation results.

The principle here is simple enough. If site investigation is conducted in a simple and inexpensive way, one can save a little money on that. Thereat, as we established before, accuracy of calculations will be, at best, ±30 %, and sometimes even essentially worse. It compels the designer to input safety factors into his structures. A reasonable safety factor at such calculation accuracy would be in the order of 2…3. Thus, as a result of trying to save on site investigation we get over expenditure of materials. Their cost will be considerably above the price of any investigation or instrumentation, because to create a structure more or less indifferent to the size of expected settlement special expensive technical solutions are necessary.

> **A guarantee of project efficiency is a well coordinated cooperation between the geologist, the geotechnical engineer and the designer.**

But there is also another way – to carry out high quality (and consequently slightly more expensive) site investigation. However, for the expensive research to be profitable for the investor, a well-fitted team comprising the geologist, the geotechnical analyst (programmer) and the geotechnical designer should work together. Only when the geologist understands what characteristics he should provide the analyst with, the analyst understands how soil behaves during laboratory testing, and the designer understands how to use geotechnical calculations, does it become reasonable to expect that the designed structure will be most financially viable and reliable at the same time. It is this principle of working together that we were able to implement in "Georeconstruction" Institute.

Use of simplified subsoil models (coefficients of subgrade reaction)

In practical design simplified models of subsoil are often used. The most simple model is the single-constant Winkler model. In this model the settlement of a point in the subsoil is proportional to the pressure in this point. The factor of proportionality (*aka* coefficient of subgrade reaction) characterizes rigidity of the "spring" positioned in each point of the subsoil. When modelling slabs, the "springs" are "smeared" in regular intervals across the square area of the slab. Not getting carried away with either the formulas or the semantics of the term "coefficient of subgrade reaction", it is possible to represent this model as a kind of spring sofa with upholstery removed and springs exposed. The analogy gives correct representation also of how this model works: where we have sat down on the sofa (applied load), the springs compressed (deformation occurred). The neighbouring springs (to which no load was applied) did not compress. Scientifically speaking, the model does not describe the distributive ability of soil. Certainly, it is rather far from reality, where, if a load is applied in any place, settlement is caused within the entire scope of the settlement trough.

To amend for the drawbacks of the elementary Winkler's model its multiple variations were developed with two or three coefficients of subgrade reaction. In Russia the most widespread is Pasternak's model with two coefficients of subgrade reaction (conversely, in the West this model is almost completely unknown). Continuing with our "sofa" analogy it is possible to say that these versions of the model correspond to a spring sofa with upholstery on top of the springs. If parameters are correctly selected Pasternak's model is capable of representing a settlement trough around the loaded area.

There is one more approach to simplify subsoil behaviour – the use of one but variable coefficient of subgrade reaction. Indeed, rigidity of subsoil can be defined very simply if you divide pressure on the area by settlement. As a result you shall obtain a field of coefficients of subgrade reaction variable through the area. The trouble with this model is that for its proper use it is necessary to know the calculation result in advance. The way out can be found in the iterative solution algorithm: first we assume the load on the subsoil, then calculate the settlement, then divide the load by the settlement (thus obtaining coefficients of subgrade reaction), then use them to calculate the structure and again obtain the load on the subsoil. All this needs to be repeated until sufficient accuracy is achieved. If you do not forget the necessity of iteration for this algorithm, everything turns out properly done, but takes a long time. If you neglect the iterations the result is quick but wrong.

> Simplified models using coefficients of subgrade reaction are hopelessly outdated.

On the whole, speaking about coefficients of subgrade reaction, it is possible to say that today these models are already hopelessly outdated. With competent selection of parameters they, at best, allow to represent, and with some degree of error at that, the elastic subsoil (*i.e.* the same model, as used in the engineering methods, with all its drawbacks and low accuracy of settlement prediction). But in contemporary programs elastic subsoil is much easier and more beautifully represented by volumetric elastic elements. These days computing power of modern machines well allows to achieve this. Using such approach it is not necessary to select coefficients of subgrade reaction and is more difficult to make mistakes about which we shall briefly talk below.

Typical mistakes made when using coefficients of subgrade reaction

Unfortunately, our practice of examining calculation results tells us that using coefficients of subgrade reaction 90 % of engineers make rather bad mistakes. It is connected both with lack of literature covering these issues, and with unwillingness on the part of some specialists to understand subsoil behaviour in detail and to look deeply into specificity of its modelling.

The First Mistake: undervaluing the second coefficient of subgrade reaction in Pasternak's model. It is difficult to tell, where the myth came from that the second coefficient in this model can be assigned in fractions of the first. If you closely look at the formulas for coefficients of subgrade reaction you will find that these coefficients even have different dimensional parameters. As a rule, when dealing with a big compressible stratum in standard units of measure (kN-and-meters or tons-and-meters) numerical value of the second coefficient should be even greater than the first. If the second coefficient is set below the first, the model degenerates into a Winkler type, and the analyst actively deceives himself, pretending to be working with Pasternak's model.

The Second Mistake: absence of beyond-the-contour areas in Pasternak's model. As we already explained, this model was created to account for such a phenomenon as the settlement trough around the zone of load application. Therefore the analysts who do not include into calculation the area outside the slab, simply do not understand, how the model works. Often this second mistake goes hand in hand with the first, and sometimes the analyst piously believes that introduction of beyond-the-contour area is of little or no value. Indeed, if you have made the First Mistake (*i.e.* in fact you are actually using a Winkler-type model), introduction of the area outside the slab will truly have no effect.

The Third Mistake: assignment of incorrect values to coefficients of subgrade reaction. As we already said, coefficients of subgrade reaction, at their best, are only just about capable of somehow representing elastic subsoil.

Therefore they need to be selected in such a way that the obtained settlement should have at least a similar order of values to settlement calculated using engineering methods. Sometimes an opinion is voiced that by means of coefficients of subgrade reaction we define settlement differential, instead of settlement *per se*. This is certainly wrong. If we were essentially mistaken about the value of settlements, we, naturally, also miscalculated its differential.

Chapter 10, explaining methods of evaluating accuracy of geotechnical calculations

The elementary rule for checking soil-structure interaction calculations

So, finally we have reached the most important information in this part of the book – the rules for checking geotechnical calculations. You have been brought calculations for the designed building (St. Petersburg codes stipulate that all calculations for the subsoil and foundation of the future building should be bound in a single volume entitled "Geotechnical Substantiation"). According to contemporary codes, for any building (if it is in any way more complex than a garden shed) it is necessary to perform a soil-structure interaction calculation. Below we shall show how it is possible to quickly and effectively evaluate the quality of such work.

First of all, it is necessary to look, whether calculation of settlement has been performed by the method of layer-by-layer summation, according to SNiP 2.02.01-83* (other engineering methods in St. Petersburg ground conditions had better be avoided). If no such calculation is in sight – safely send the document back to be finished properly. Even the most sophisticated numerical techniques should always be collated with an engineering method. Absence of such an elementary calculation is simply a display of laziness, whatever scientific terminology is used to camouflage it. It is of course permissible to criticize accuracy of such a calculation, but its results should be there – this is final, peremptory, *non est disputandum, etc.*

Now let us take a shufti at the beautiful pictorial display of soil-structure interaction. From the text we find out which model the analyst employed. If it is coefficients of subgrade reaction, elastic subsoil, "Coulomb-Mohr", "Drucker-Prager" or something similar, then the settlement value, as we already explained, should not deviate a great deal from the value obtained through layer-by-layer summation. Reasonable logical differences can only be present if more sophisticated hardening soil models were used. If your analyst employed such models (for example the Hardening Soil Model (HSM) of PLAXIS or the viscoplastic model of *FEM models*) then, in order to check such results, you can make use of our advice contained in the following section.

If no complex models were used, but the calculation results are significantly different from the method of layer-by-layer summation required by SNiP, it means that the calculation is mistaken.

So, if the text does not mention complex nonlinear soil models then it will suffice simply to compare calculation according to the method of layer-by-layer summation and settlements obtained through computer calculation. Inaccuracy of about 30 % is possible (as the accuracy of the layer-by-layer summation itself is no better than ±30 %). But if it is in excess of the 30 % – safely send the analyst away to read scientific books and perform a recalculation. And it would be even better to hire a specialist who has already read the books and is not capable of making such elementary mistakes.

Simple rules of using complex models

Now we shall examine a more complex situation when the calculation was performed using advanced nonlinear soil models, for example, the viscoplastic model of *FEM models* or the Hardening Soil Model of PLAXIS. Using these models the analyst can rightfully declare that they represent soil behaviour more correctly, than the elementary engineering methods of calculation. This is, indeed, so, but we need to remember that complex models require equally complex sets of parameters. The model will reflect soil behaviour only if those parameters are assigned properly.

Therefore, evaluating results of calculations according to complex models, it is necessary to ask oneself the following question: where did the numerous parameters for these models come from? They are absent in the standard geology used for our domestic practice. For example, where will the analyst obtain the triaxial modulus if no triaxial tests were performed? Unfortunately, in many cases it will appear that the parameters were taken "out of the blue". In this case one should rightfully expect a somewhat "bluish" result of calculations. Such approach discredits the whole idea of using complex models. This, unfortunately, is promoted also by some software developers. Clearly, they need to sell their produce, therefore programs often assign the missing parameters "by default". Accuracy of such assignment is absolutely arbitrary. It is all the same as defining geological parameters by means of the random-number generator. As a result, geotechnical engineering from being the exact science degrades into reading teacups, only with application of computer facilities.

Complex soil models may not be used without meticulous soil tests.

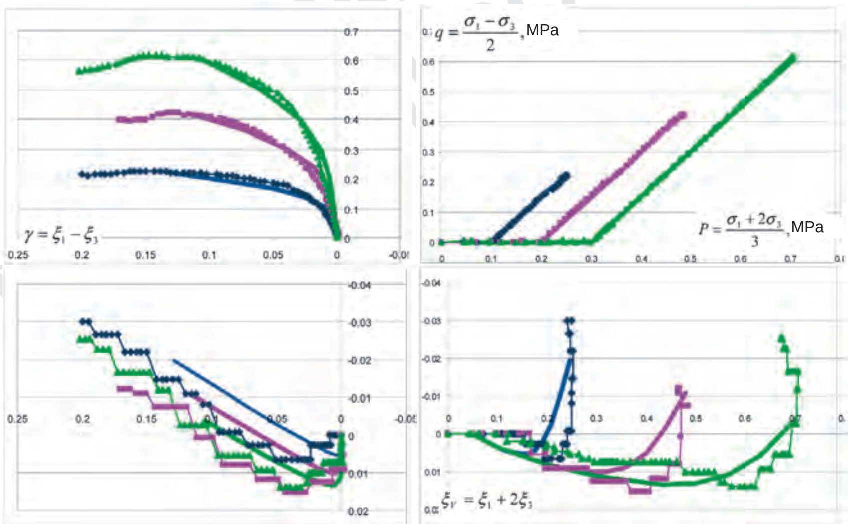
! Hence the first simple rule of using complex models: ***without meticulous site investigation coupled with detailed laboratory tests (including triaxial) complex models of soil mechanics may not be used***.

Let us assume we have these long-awaited high-quality site investigation results in which requirements of codes and standards have finally been fulfilled, and the appropriate laboratory tests have also been done. How to check whether the analyst correctly assigned the model's parameters?

The model should describe a laboratory test.

The second simple rule of using complex models: ***the model should fairly accurately describe results of laboratory soil tests.*** The geotechnical substantiation ***should contain a proof thereof;*** the result of modelling compression tests should coincide with compression curves; modelling triaxial tests should give the same curve, as in laboratory tests. If such comparison is missing from the geotechnical substantiation it is possible that the analyst is not able to select parameters of his model correctly, which means he is not able to use it. At this stage laymen will naturally be eliminated as they will not be able to correctly simulate laboratory tests, because they poorly understand how it is done.

Once a PLAXIS calculation was submitted to us for expert examination. Our question as to the compression curve adopted in calculation generated a response from the authors, which sounded something like: “Compression curve? What’s this then, eh?” This, Dear Reader, is a good and proper failure on a college soil mechanics exam.



Example of selecting parameters of nonlinear model according to triaxial test routine. Points represent test data, continuous lines – modelling of the test.

Certainly, there would be charlatans who would use an opportunity to feed you phoney results of comparison with laboratory tests, having forged a corresponding curve by hand. Such danger, basically, is always there, even calculation results as such can be drawn artificially using a graphic editor. Your sole rescue here is that a self-respecting specialist would never do such a thing, and a forger will always reveal his cloven hoof, one way or another.

Calculation results should be compared with *in situ* measurements.

! And, finally, the third simple rule of using complex models: ***results of calculations should be compared with local in situ monitoring results***. This simple rule is really quite simple also to observe. It took us about two years to collect our settlement monitoring data and to test results of calculation using our own nonlinear model. Results of comparing calculations with monitoring are given above in Chapter 9. Only after that does it become possible to confidently use a complex nonlinear model in design practice. Without such self-training one really should not use complex models. Here the analogy to usual work tools is pertinent – the more complex they are, the more time you need to get the hang of them. Inept handling of a hammer will, at worst, get your finger smashed or shatter your window. Having saddled a tractor or a skip-loader without a due skill, it is possible to wreak a significantly more noticeable havoc.

A geotechnical substantiation should contain results of comparing calculations obtained through the chosen model with *in situ* tests, proving that the model's application allows to receive good accuracy of calculations. Importantly, the test data should come from a similar type of soil.

Complex models should only be used by specialists in the area of geotechnical calculations.

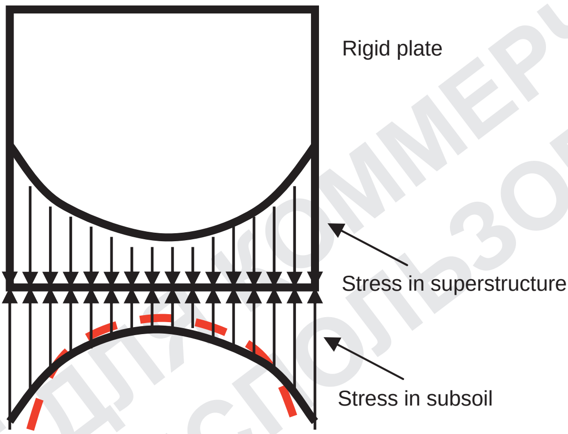
It is obvious that the third rule can only be observed by professionals specializing in the field of geotechnical calculations as other specialists will have neither time, nor requisite knowledge for detailed testing of numerical models. Therefore last and the most simple rule of using complex models sounds like this: ***complex models should only be used by specialists in the area of geotechnical calculations***. There are, sadly, only very few such experts in the world. As a rule, they are people who are themselves engaged in development of numerical calculation software programs or nonlinear models of soil mechanics, which is quite natural, as profound knowledge of models construction and numerical programs often urges one to improve them. Therefore when choosing the analyst the preference is best be given to an expert who is himself engaged in scientific activity in the field of numerical calculations. Such experts understand better than others features of complex programs and nonlinear models and will be in the position to do calculations competently. Certainly, science, as we already said, should be organically combined with practical design.

Chapter 11, explaining the main effects revealed in soil-structure interaction calculations

Historically there always was division of labour in design of buildings: the superstructure (sometimes, including the foundations) was the domain of the structural engineer, whilst the subsoil was dealt with by the geotechnical engineer. Such specialization is quite reasonable: the structural engineer deals with artificially created objects – reinforced concrete, structural steel and so on, whereas the geotechnical engineer works with a natural environment – soil. The mechanism of interaction of structural and geotechnical engineers in the Russian (and international) realities is usually like this. The structural engineer relates to the geotechnical engineer the information on loads transferable from the building onto the subsoil. Thereat concentrations of loads, irrespective of whether they are obtained manually or as a product of solving a finite element problem of superstructure calculation, are figured out without taking into account deformability of the subsoil. Differently put, calculation of loads transferable onto the subsoil is made as if the building were to stand on a kind of a rigid unyielding table. The geotechnical engineer receives these loads and applies them as flexible (!) to the subsoil which he models, using contemporary achievements of soil mechanics. Therefore we ask: what kind of subsoil do we have – absolutely rigid (as the structural engineer thinks) or flexible (as is rightly believed by the geotechnical engineer)? Also, what building do we have – flexible (as modelled by the geotechnical engineer) or of final rigidity (as fancied by the structural engineer)? The way out of this conundrum seems rather simple: it is necessary to calculate the building together with its subsoil. This requirement has been for a long time included into the Russian normative documents.

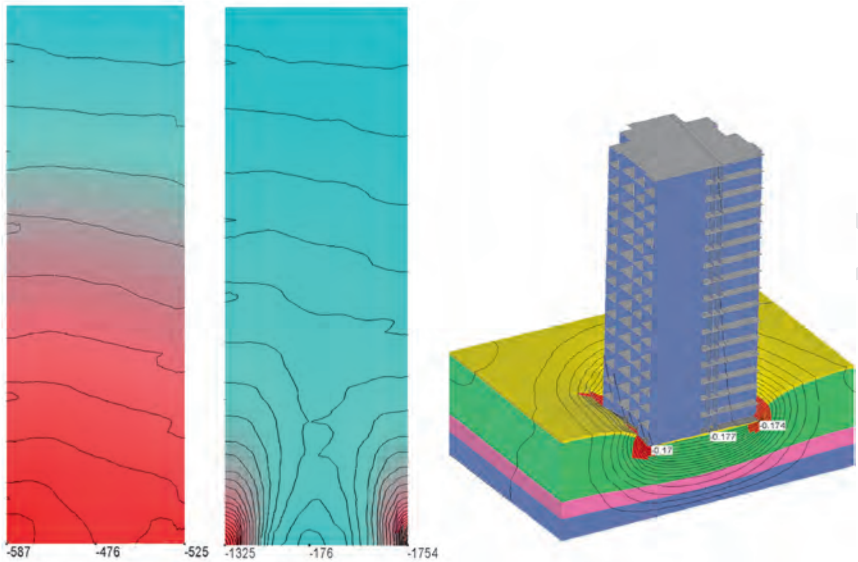
It is pleasant to note that in the field of soil-structure interaction Russia is not behind other countries, but, on the contrary, occupies one of the leading positions in the scientific world. A recognition of merits of Russian scientists is that they head Technical Committee 207 "Soil-Structure Interaction and Retaining Walls" of ISSMGE (International Society for Soil Mechanics and Geotechnical Engineering); the committee is chaired by professor V.M. Ulitsky.

Let us examine the main effects revealed in combined calculation of a building's structure and its non-linearly deformable subsoil. These effects have been known since the very inception of soil mechanics. Their "novelty" is, more likely, a psychological problem which arose because of traditional dissociation between calculations of buildings and their subsoils. In any textbook on soil mechanics we shall find the well-known epure of contact pressures under a rigid plate.

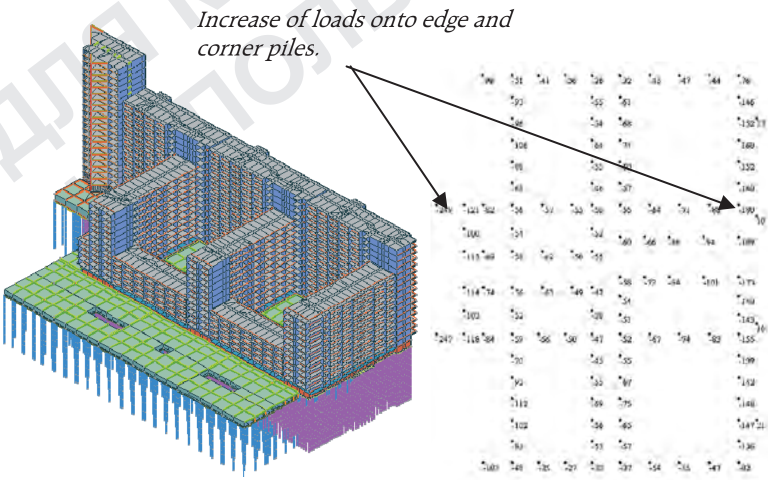


Theoretical stress in elastic half-space.

For the theoretical solution of this elastic problem it has a parabolic appearance stretching into infinity in the edge zones. For real soils it has the characteristic saddle-like appearance which changes at significant pressures when the building approaches loss of stability. It is obvious that loads in the plate itself will be the same. Therefore it should not be the cause for any surprise that when the plate is replaced with a real structure, vertical normal stresses in edge zones strongly increase. The increase manifests in the zone whose height is approximately equal to the width of the building.



Loads (kN/m) in a transverse wall of a building on natural subsoil: on the left – according to separate calculations, without taking the subsoil into account (the loads simply increase towards the bottom); on the right – according to soil-structure calculations, in which concentration of loads in edge zones is observed.



Distribution of loads in the piled foundation according to soil-structure interaction calculations.

This law is characteristic for buildings constructed both on natural subsoil, and on piled foundations. In a pile field the effect of building's rigidity leads to a loads increase on the piles in the edge zones and unloading of the piles in the center of the building. This well-recognized fact is confirmed by numerous measurements.

The soil-structure interaction effect: loads concentrate in the edge zones (both in the superstructure and in the piles).

Briefly stated, the main effect of accounting for spatial action of the subsoil and its interaction with the superstructure is rather simple. The subsoil always tries somehow to "bend" the structure (even when loads from the latter are close to evenly uniform). The structure in turn tries to prevent this bend as best it can. As a result, additional stresses appear in structures, which it is necessary to account for in structural design. This is especially important for buildings and structures with complex layouts: their redistribution of stresses can lead to a rather peculiar overall "load-play".

Given all the simplicity and clarity of the described soil-structure interaction effect, practical solution of specific problems is complex enough. In fact what the structural designer needs is not the abstract knowledge of the effect's presence, but its clear numerical expression. The effect itself is revealed even in simple elastic calculations. However, as we could well see above, accuracy of such calculations is rather low and absolutely insufficient for structural design. Therefore, practical applications of soil-structure calculations are directly connected with use of complex nonlinear soil models which make geotechnical calculations more accurate.

Unfortunately, in reality some analysts try to pass off their calculations "on springs" as soil-structure interaction. As we demonstrated in the previous chapter such approaches have become hopelessly outdated. Springs cannot adequately represent the subsoil, and, most importantly, cannot correctly describe its settlement differential which is the most essential in superstructure calculations. If such simplified approach is adopted, the structural designer should be explained that accuracy of subsoil deformation calculations being low, loads in the superstructure are calculated with precision of about 50%. This message hitting home, any sane designer will simply throw away such calculations and assume the double factor of safety in his structures (at the investor's expense, of course).

Therefore a competent account of soil-structure interaction requires use of complex nonlinear soil models, for which simple rules of application are given above.

Chapter 12, explaining calculations of cofferdams and underground structures in congested city environment

The subject of this chapter is very important for St. Petersburg. In order to develop, the city needs underground parking facilities, traffic interchanges, warehouses, *etc.* In geological conditions of St. Petersburg underground construction is a die-hard challenge. Calculations of retaining structures prove equally difficult.

For construction in congested city environment it is absolutely not enough to just calculate stability of the cofferdam: it will most probably not fail, but it will most definitely move or deflect, the result of which will be the loss of an adjacent existing building. It is also necessary to perform calculations of the cofferdam's deformation and settlements of surrounding buildings.

In such calculations it is necessary to account for a lot of important things: works schedule, rigidity of the cofferdam, soil excavation rate, peculiarities of soil behaviour *vis-a-vis* rate of works, *etc.* Additionally, you must consider the history of loads on foundations, you must model the existing buildings and predict their expected additional settlements at each stage of works. The total predicted settlement should not exceed permissible tolerances (as a rule, for historic buildings the settlement tolerance is no more than 2…3 cm).

The simplified linear models here are absolutely helpless. And, as a rule, using the elementary ideally elastoplastic model with criterion of strength according to Coulomb-Mohr ("the Coulomb-Mohr model") in such calculations is also impossible. The latter incorrectly describes soil action under removal of load during excavation; as a result the bottom of the pit "pole-vaults" to incredible heights, at times even pulling existing buildings in its wake. In reality, however, existing buildings settle downwards. Among calculations submitted to us for expert examination there was one curious case when the author of the calculation received a rise of adjacent territory at the radius of 50 m around the underground structure, and besides even went so far as to suggest measures to counteract this mythical uplift.

A typical mistake when using PLAXIS software is wrong assignment of ground water level. It is enough simply to forget to perform one step (to draw the altered position of ground water in the foundation pit) – and the pressure upon the cofferdam miraculously drops almost 2 times! When this happens, the program does not alert the user of his oversight and simply proceeds to calculate the foundation pit filled with water. Alas, one often comes across this mistake in evaluation of calculations. As contractors will obviously object to underwater concreting, it would be necessary to equip the authors of those calculations with scuba-diving sets to perform rebar fitting under several meters of water.

When calculating cofferdams in conditions of congested city environment the best results will be obtained using complex nonlinear soil models. It is clear from experience that deformations of cofferdams occur over time, therefore it is desirable to model bulk excavation process with special rheological models in view of real timeframe of work stages. Soil models should be tested by means of comparing calculations with *in situ* test data. For this purpose we had several test pits organized in various city districts, which were used to give our viscoplastic soil model incorporated into *FEM models* software a thorough testing.

Bulk excavation should be modelled with the help of rheological dependencies, with account of realistic timeframe of works.

Fully instrumented deep excavation test pits in St. Petersburg with detailed monitoring of soil behaviour.

Do not hesitate to contact specialists in the area of geotechnical calculations.

On the whole, calculating cofferdams is impossible without complex models of soil mechanics, and analysing these calculations you may make use of the above simple rules of how to handle complex geotechnical models. And remember that the main rule is not to hesitate to contact specialists in the area of geotechnical calculations.

Part Three

A tour through geotechnical construction

Chapter 13, explaining the choice of contractor

Let us assume that you have a design prepared by a True Designer which means that you also have a Geotechnical Substantiation.

In this Geotechnical Substantiation certain sparing technologies were selected for the construction of your project. Now it is finally time to start building.

Let us further assume that the True Designer prompted you to meet prospective contractors and now you have to make your choice.

Here you should follow the same guidelines as when choosing a designer. Do not trust words. In words everybody can do everything. Take time to visit the contractor's sites and to go to his facilities. Check whether machinery and equipment belong to him and not, as it were, to Marquis of Carabas.

We advise you to be on the alert if your prospective contractor immediately starts streamlining your design. Sometimes such ameliorations sound attractive. But more often than not it is the same robbing the project of safety we have already discussed.

Just imagine: you have come to a chemist's with a prescription and a pharmacist who has known you for two full seconds already declares that the doctor was wrong and you need some other medicine. It is far better to listen to a second opinion of another doctor than to follow advice given by a pharmacist.

The designer and The contractor are going to cooperate till the completion of construction. The client concludes an agreement with the designer for author's supervision so that the author of the design could control that the works carried out on the construction site are true to the drawings.

In addition to the above quite often the contractor or the client ask the designer to supervise the construction in order to solve all the arising technical problems.

To make a correct choice of a contractor it is very useful to have a look at his completed projects. It would be quite wise while doing so to pay special attention to the adjacent buildings. If they are covered with screens featuring drawn facades (like those works of art we could admire for a decade next to the Nevsky Palace Hotel), then the contractor somewhat exaggerates his proficiency. Sometimes freshly renewed appearance of adjacent buildings should also look suspicious.

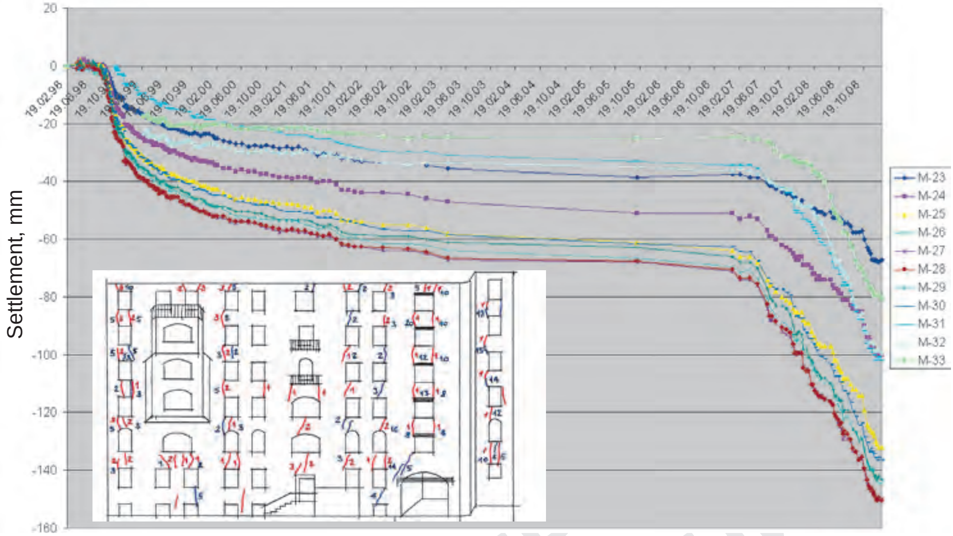
When selecting a contractor do not merely trust his words. Have a look at his projects and his facilities.

Ask the contractor to show you the results of settlement monitoring of the adjacent buildings. Rest assured, if everything is as fine as the contractor is telling you he will show you these with pride. If, however, awkward silence ensues, then the result was not so good.

Here is a recent example. The famous site – suspended construction in the centre of the city. First, foreign contractors destroyed two listed mansions and caused a 7 cm settlement in the adjacent building, one of the largest historic tenement houses in St. Petersburg. A decade later construction works were resumed and brought the settlements of the tenement house down to 15 cm.

To our greatest surprise this project is now presented as an example of successful underground construction in the centre of the city. Well, there is certainly a cause for celebration – at least one of the adjacent buildings survived!

There even were some so called "renowned experts" who claimed that nothing bad had happened: only one wall had settled down by 15 cm. A civil engineer might as well give up here: truly, to call oneself a professor does not necessarily mean to be an engineer.



Settlements of the house on Ligovsky Pr. from 1998 formed during two stages of construction works and a view of cracks formation on the courtyard façade (blue colour represents cracks formed after the first stage, red colour – after the second).

Chapter 14 –
explaining geotechnologies

And so, the building works are in progress. Let us consider the geotechnologies used in modern construction. But prior to this let us arm ourselves with several useful rules that are important for St. Petersburg soil conditions and the condition of the existing buildings.

The First Rule. *A priori* safe technologies do not exist. Each method should be adjusted to soil conditions of our region. Its impact on the subsoil should be instrumentally measured by the geotechnical engineer. This procedure is called *approbation*. If a certain geotechnology produces negative effect a geotechnical engineer should firstly determine the reason for this and if possible minimize it, finding sparing technological regimes. This procedure is called *adaptation* of geotechnology to ground conditions of a construction site.

The Second Rule. No geotechnology should be used without monitoring. This will be further discussed in Chapter 16.

The Third Rule. Rush work and geotechnology are two incompatible things. Soils are natural environment and the nature does not put up with violence. Intensive impact on soil leads to its remoulding, whereby soft soil turns into heavy liquid. Besides, rush work always tends to violate sparing technologies.

> There are no *a priori* safe technologies.

The Fourth Rule. In the centre of the city one cannot hammer piles. One could use vibratory hammers but this is highly dangerous (even using a high-frequency hammer). Pre-fabricated piles can only be pushed in.

The Fifth Rule. Pile pushing should be carried out with maximum care, limiting oneself to two-four piles per day within the 10 m restriction zone in adjacency to existing buildings.

No geotechnology should be used without monitoring.

The Sixth Rule, derived from the first one: bored piles are not a universal panacea. There are technologies allowing to produce 10-12 bored piles per shift. But according to the experience, this technology is safe for the adjacent buildings when the number of bored piles is limited to 2-4 per shift (see Rule No. 3).

Just 15 years ago the arsenal of pile technologies consisted only of driven and pushed piles. Today on the Russian market a lot of western technologies of bored piles are presented. Quite often different names conceal quite similar technologies. To facilitate your surfing in this abundance of methods, we put them all in a table. We group different technologies based on the method of their production which are not many. If you come across a geotechnology with an exotic name ask how the piles are made – and you will find it in our table. We also graded different pile types on the scale of 1 to 5 (a typical grading system used in Russian schools) based on our extensive experience of monitoring in St. Petersburg.

Typical outsider is the **method of pile hammering**. It is prohibited to use this method in the area closer than 20m to adjacent buildings, and it would be better not to use it at all in the blocks of existing buildings. However, there are no limitations on using this method in "greenfield" conditions. It is only required to control the selection of proper pile-driving equipment. If piles are driven incorrectly they may be destroyed during hammering. When the next pile is being driven the previous ones may be uplifted, dramatically losing their bearing capacity.

Rush work and geotechnology are incompatible.

Sometimes a pile field may turn into a pile forest for which a pile lumberjack will be required. In this case someone surely made a mistake: either the site investigation geologist, or the designer, or the contractor who opted for selecting the pile driving method. Did he correctly choose the weight of the hammer and the hammer impact? Should he perhaps have drilled pilot boreholes?

Pile forest instead of pile field.

On the subject of **vibration driven piles** suffice it to say that they are no less dangerous for the existing buildings than the hammered piles and are also more difficult to produce.

In the 1980s **pushed piles** were considered to be a guarantee for safe construction within a city. It was believed that pile-pushing is accompanied by a beneficial soil consolidation (without any dynamics, if you will). Perhaps only wise Boris Ivanovich Dalmatov, our Teacher, was sceptical. He warned that in clay consolidation is replaced by over-kneading with lateral displacement. Indeed, try to consolidate jelly in a cup by poking a tea spoon in it. This simile, albeit a little crude, is quite appropriate.

If we are to use a scientific language, St. Petersburg soft soil is a disperse structured system, consisting of the disperse fraction (clay particles) and dispersion media (water), immobilized in pores of structural framework formed by clay particles and water bound by these particles. In scientific literature such media are called “gels” (another transcription of which is “jelly”).

Pile pushing rig УСВ120 М.

Table

Piled Foundations Technology

Place of Fabrication	Method of Construction	Method variation	Examples of equipment	Disadvantages	Safety of existing buildings	Safety grade
1	2	3	4	5	6	7
A. Prefabricated piles	A.1. Hammer driving	A.1.1. Fixed hammer drop height	Diesel hammers, hydraulic hammers	Strong hammering impact	Green Field	1
		A.1.2. Variable hammer drop height	Junttan, etc			
	A.2. Vibratory driving	A.2.1 High-frequency	ВШ	Strong vibratory impact	Green Field	1
		A.2. High-frequency, non-resonance	ВШ 402, Vibrofoncer, Tünkers, etc			
	A.3. Pushing (Jacking)	A.3.1. From loading platform	Trust 101 Rig	Uplift of soil, piles, existing buildings; settlement on account of heavy-duty machinery	2-4 piles a day in restriction zone	3-
		A.3.2. Heavy-duty self-propelling rig	УСВ120, УСВ120М			
B. Cast in place piles	B.1. Soil replacement	B.1.1. Under bentonite protection	Franki	Bad mix quality=faulty pile=buildings' settlements	Mix quality control	5-
		B.1.2. Casing protected	Bauer, Cazagrande, Junttan, Double Rotary	Overdrilling= buildings' settlements	Control of soil "plug and filling with water to the brim	5

End of Table

1	2	3	4	5	6	7
B. Cast in place piles	B.2 Cartridge drilling	B.2.1. No spoil (screwed piles):		Uplift of soil and existing buildings; damage to raw piles	2-4 piles a day in restriction zone	3
		a) Forming smooth surface pile shaft	Fundex			
		b) Forming ribbed surface pile shaft	Franki "Atlas" Rig			
		B.2.2. With partial spoil (continuous flight auger)		Uncontrolled overdrilling= buildings' settlements	Green Field	1
		a) whole pile length auger (CFA)	Multiple manufacturers			
		б) auger assembled of adjustable units	DIYs			
	B.3 Special borehole formation	B.3.1. Pneumatic impact borehole	Pneumodrill	Strong dynamic impact	Green Field	1
		B.3.2. Electrohydraulic widening of borehole	Electrohydraulic drilling			
		B.3.3. Widening of pilot borehole	DDS	Uplift of soil and existing buildings; damage to raw piles	2-4 piles a day in restriction zone	3

Pushing piles into this jelly leads to surface uplift, squeezing out of previously driven piles and even neighbouring buildings. Sometimes piles rise by 25-30 cm, and buildings rise by 3-5 cm. It is surely possible to drive the piles back, but the risen buildings start plunging back on their own, because their subsoil had been over-kneaded (remoulded).

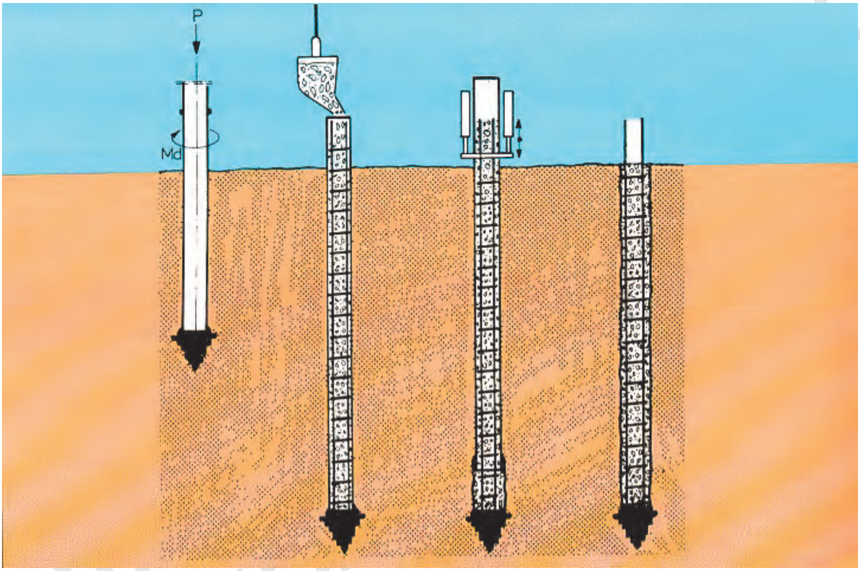
Within a 10 m restriction zone the method of pile pushing should be used with extreme care.

Pile pushing is carried out by heavy-duty rigs weighing up to 120 tons. The movements of this rig along the building may provoke the latter's settlements.

To sum up: within a 10 m restriction zone the method of pile pushing should be used with extreme care, producing no more than 2-4 piles per work shift. This will allow some relaxation of the strains formed in the soil around the pile.

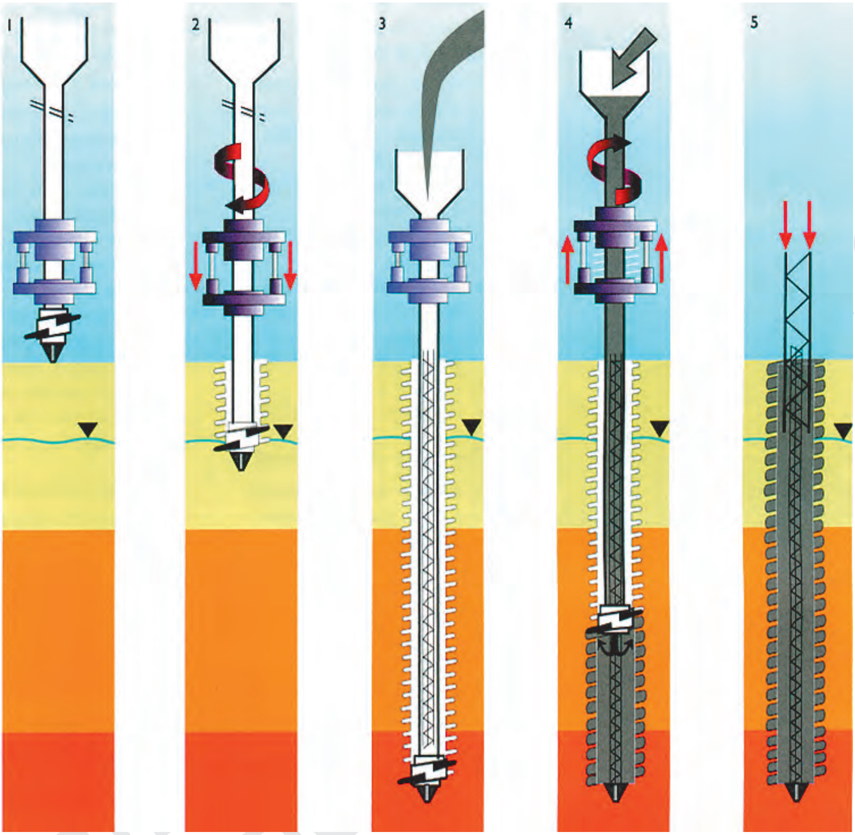
The technology of ***bored piles without soil displacement*** is very similar to pushing as far as its impact on soil is concerned. The soil is displaced with the same consequences. The pile body is formed by pushing and (or) by screwing a smooth cartridge with a sacrificial threaded tip into the soil. Inside the cartridge a reinforcement cage is placed, the borehole is filled with concrete after which the cartridge tube is removed.

When after applying Fundex technology the adjacent heavy five-stored building rose by 40 mm, neither Russian, nor foreign specialists could believe it. However, it was hardly a miracle. The contractor constructed 153 piles in just 19 days, moreover the work front was moving towards the building pushing forward a wave of heave. Within following 3 years the adjacent building settled by 10 cm, and the new one developed differential settlements which is quite unusual for a building on a piled foundation. The reason for the differential settlement is the effect of pile heaving. And please note: it is impossible to redrive a bored pile after it has been pushed up. Besides, there is another burning question: what happens to the raw, unset pile when it is being heaved by several centimetres? Therefore, a thoughtful client should hire a geotechnical engineer to provide a special supervision service for the works.



Construction of Fundex piles: consecutively – screwing in a cartridge in the form of a tube with a sacrificial tip, installation of a reinforcement cage and concreting, removal of the tube.

The supervision should start from the method statement revision, namely from the map of movement of the pile rig. It should be moved around the whole site quite a lot so that the soil is given a possibility to relax. Then, the geotechnical engineer should survey pile heads level – to make sure they have not started to rise. Each risen pile should be tested for its integrity (to check if the shaft hasn't been "torn").



Atlas piles construction: 1 – preparation;
2 – screwing in a cartridge by means of vertical force;
3 – inserting inner reinforcement cage inside the cartridge;
4 – concreting while removing the cartridge;
5 – inserting outer reinforcement cage.

It is necessary to control that no more than 2-4 piles are produced per working shift within a 10 m zone of adjacency to the existing buildings, and that a new pile is started not earlier than 3 days after the previous one (so that it could accumulate some minimum strength).

Productivity of the modern technologies – 10 piles per day. Safe intensity of works – 2-4 piles in the zone of adjacency to existing buildings.

Strict following the above recommendations of the geotechnical engineer will make the technology relatively safe for the adjacent buildings.

But, alas, its “firing rate” will be lost, and that is what the contractor was boasting about when he promised to produce 10-12 piles a day.

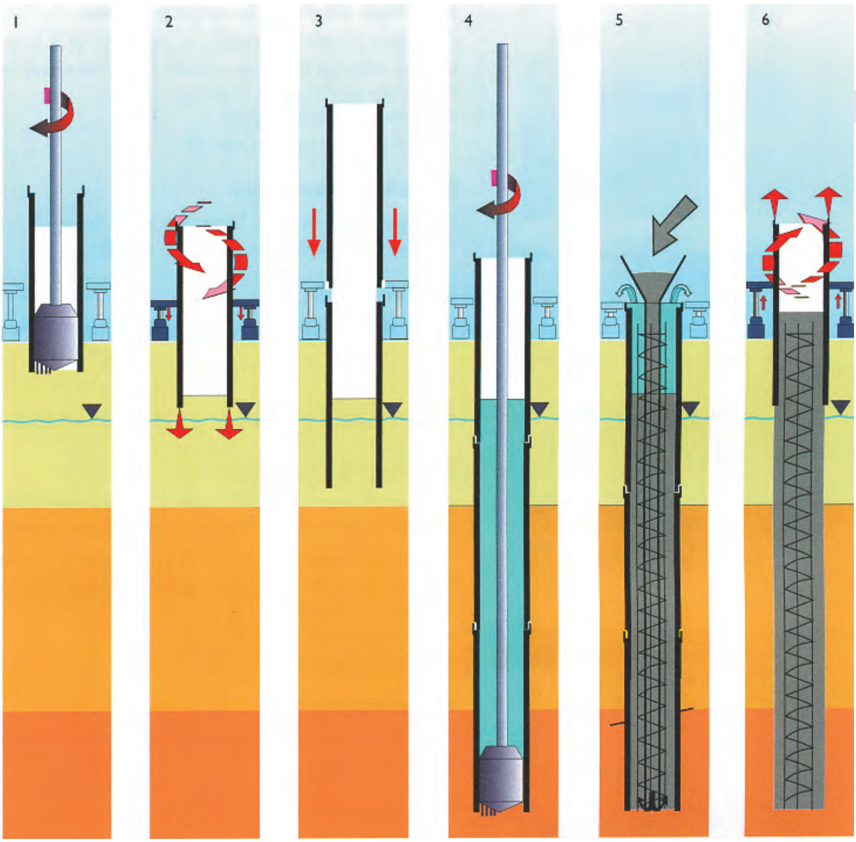
As a result the technology of pile pushing and bored piles with soil displacement should be graded no higher than 3 (that is to say, satisfactory). These technologies are productive when unsafe and safe when non-productive.

Probably, we should not dwell on exotic technologies such as electrohydraulic discharge-impulse. Surely, discharges can vibrate concrete mixture, and compact sands, but not our *jelly*. Weak discharge is useless, and strong discharge is dangerous. This technology has not been adapted to the geological and engineering conditions of St. Petersburg. Contractors themselves are aware of this fact and use the discharge more like an advertising gimmick. In fact, on several sites we observed that the discharger was not even powered.

By the way, it is very useful to know how to distinguish an advertising trick from a genuine feature of the technology. Quite often the contractor claims that the piles built according to his method have a “higher” bearing capacity. But when one tries to inquire as to “higher than what”, the answer is – “higher than he thought”.

For example, a company was advertising its bulb-end piles, boasting of their high bearing capacity. Later it turned out that usual piles without the bulb have just the same bearing capacity.

Two technologies from our table were given the highest grade. They use ***soil replacement boreholes***. Thus the pushing effect is eliminated. However, another question arises: how to hold the borehole walls in place? One of the technologies uses a special **bentonite solution** for this purpose.



Casing protected method of pile construction:
1,2 – inserting of the first link of the casing; 3 –connecting the next link of the casing; 4 –drilling down to the design level (to prevent excessive soil extraction the drilling is carried out either with a preserving a special soil plug or under the protection of water column); 5 – installing a reinforcement cage and concreting with a vertically moved tube; 6 – withdrawal of the casing.

Bentonite is a special clay that can transform into a gel very quickly even when the concentration in water is low. It is a little heavier than water but does not discharge water into soil and is capable for a certain period of time (several hours and even days) to hold the walls of the borehole in soft soils.

Another technology uses a casing to fix the walls of the borehole. A casing is carefully inserted by being oscillated into the soil (twisting it right and left with simultaneous downward thrust). Soil is extracted from the hole. While doing this it is necessary to control that there always remains a soil plug at the bottom of a borehole and the hole itself is filled up to the brim with water. These technologies are not the speediest: it is possible to produce only 2-3 piles per day. But considering that all technological regimes are observed they provide the best outcome in terms of safety of the neighbouring buildings.

Another rule that is applicable to bored piles, among the others: the main enemies of all modern geotechnologies are homebred innovators. Only here the seemingly safest technology of casing protected pile construction was "rationalized" in such a way that it led to destruction of the neighbouring buildings on several sites. They just dispensed with filling the borehole with water (why make further mess?!) and so uncontrolled excessive spoil extraction (commonly called "overdrilling") occurred.

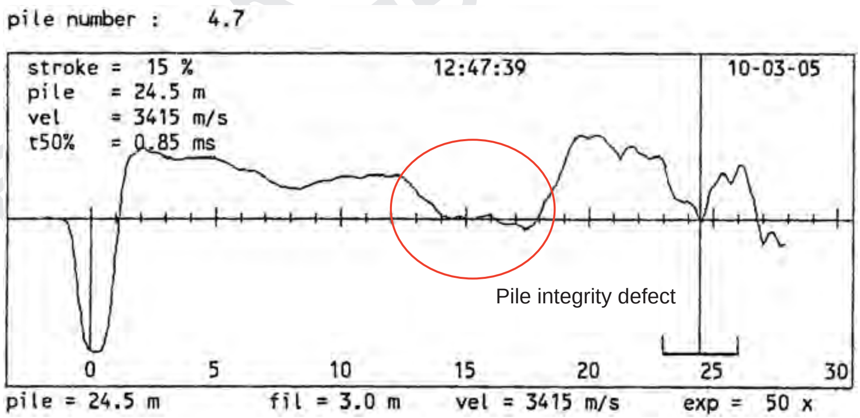
It would seem that there could be nothing dangerous in constructing casing protected bored piles almost in the "greenfield" conditions, what with the existing buildings situated further than 15 m from the site? What can be easier than to insert the casing a little bit ahead of soil extraction from the hole? You can do one single high quality and really safe pile per work shift. But it is too dull for our people. They are paid by the number of produced piles. One can speed up the process by disregarding the safe technology and extracting soil below the undercut of the casing. Then the casing will fall into the borehole on its own and thus it will be possible to produce two or even three piles per shift. But the neighbouring buildings will settle in the process. So, to what extent did they have to violate the technology if the buildings that are situated 15 m away from the site settled by several centimetres?!

A common rationalization of the method of pile construction using bentonite slurry is replacement of the latter by the so called “drilling slurry”. The technology stipulates the accompanying use of the whole bentonite plant on site: the bentonite slurry is supposed to be constantly strained of spoil which finds its way therein.

The reliable way to check quality of bored piles is to do pile integrity test.

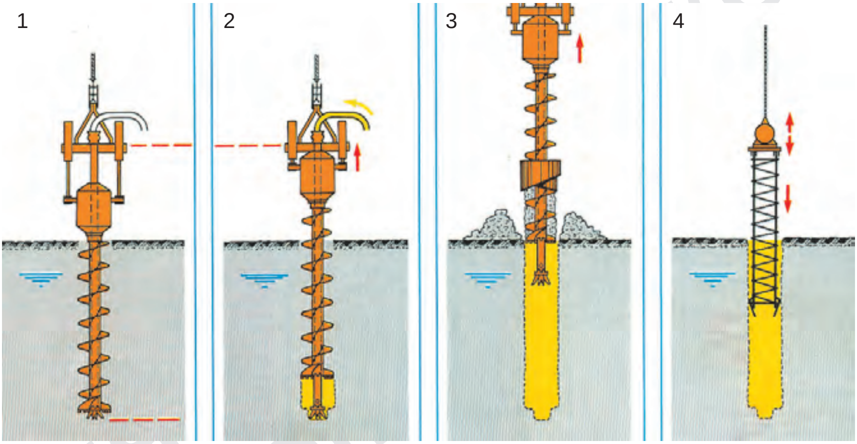
Home-grown innovators save money by filling the borehole with water, instead of bentonite. It swirls inside the borehole during drilling and the resulting liquid mud is proudly called “drilling slurry”. However, no matter what you call it, this slurry does not have the properties of bentonite, and the resulting pile might turn out faulty, dangerous when loaded.

Truly, there is no western geotechnology which would not be ruined by local artisans.



Pile integrity test.

Several times the present authors observed the following *know-how*: local craftsmen apply nice plaster finishing to pile heads before handing the pile field over to the Client. And what is, may we ask, below? Can the Client check the quality? Fortunately, there is a way to expose the artful dodger. For over a decade we have been testing constructed piles. The method we use has become obligatory in road construction and is known as "integrity test". The principle behind the test is quite simple: a pile is hit with a hammer, the sound wave travels through the pile shaft and reflects from its toe or a defect (necking, rupturing or cracking).



CFA method of pile construction:
1 –the auger is bored into the soil down to the design depth;
2 – the aperture is prepared for pouring the concrete mixture;
3 – borehole is concreted while extracting the auger;
4 – reinforcement cage is vibrated into the pile shaft.

Among all bored piles technologies the champion of destroying the neighbouring buildings is the ***CFA method.*** In 1998 two listed buildings near Moscow Railway station were totally ruined, in 2007 Muruzi House was partially damaged. In 1998 we managed to save Pertsov House (44, Ligovsky Pr.) from this technology. We proved its danger to the foreign contractors (they believed us only when buildings Nos. 26 and 30 along Ligovsky Pr. were destroyed and consequently demolished). The danger of the technology is in uncontrolled spoil extraction (uncontrolled overdrilling) while screwing the auger in. It is easy to insert the auger into soft soil, but to get into the underlying harder layers some pressure is required. The technology does not allow for this. Try to put a screw into a wall just with your fingers! As the result the auger rotates in one place extracting soft soil to the surface, like a meat-grinder, and producing subsidence trough around the pile. We have published a considerable number of works on CFA technology. It is a shame that 9 years later ignorant designers stepped on the same rake again. It seems that in Russia rake-walking is a kind of a national sport. We will discuss this, Dear Reader, a bit later, in Part Five of this book.

Neither the description of a technology nor positive experience of its application in Moscow, London or Paris, where the soil conditions are much better than in St. Petersburg, can vouch for its safety. It seemed that there was nothing dangerous in the ***Double Rotary*** technology, which involves soil extraction under protection of a casing, screwed into the soil in the opposite direction. Unlike CFA technology, due to the casing the auger cannot overdrill. However, in reality settlements of the adjacent buildings reached 3 cm. It turned out the auger was getting stuck in the casing. It was extracted to get cleaned and at this moment, working as a piston, it sucked soft soil into the casing! This example also proves that without prior testing and adaptation to local soil conditions no geotechnology can ever be believed to be safe enough.

Chapter 15 – explaining geotechnologies further, this time for the purposes of underground construction

In the previous chapter we discussed the extent of damage untested technologies of pile construction can cause the neighbouring buildings. But it is nothing compared to destructive capabilities of thoughtless underground construction. Construction within the urban area can cause such serious damage to adjacent buildings which could lead to evacuation of residents. Mistakes in the underground construction may lead to such magnanimous consequences that nobody will even have time to leave the building which will collapse in one moment.

Infinity Tower pit collapse, Dubai (2007).

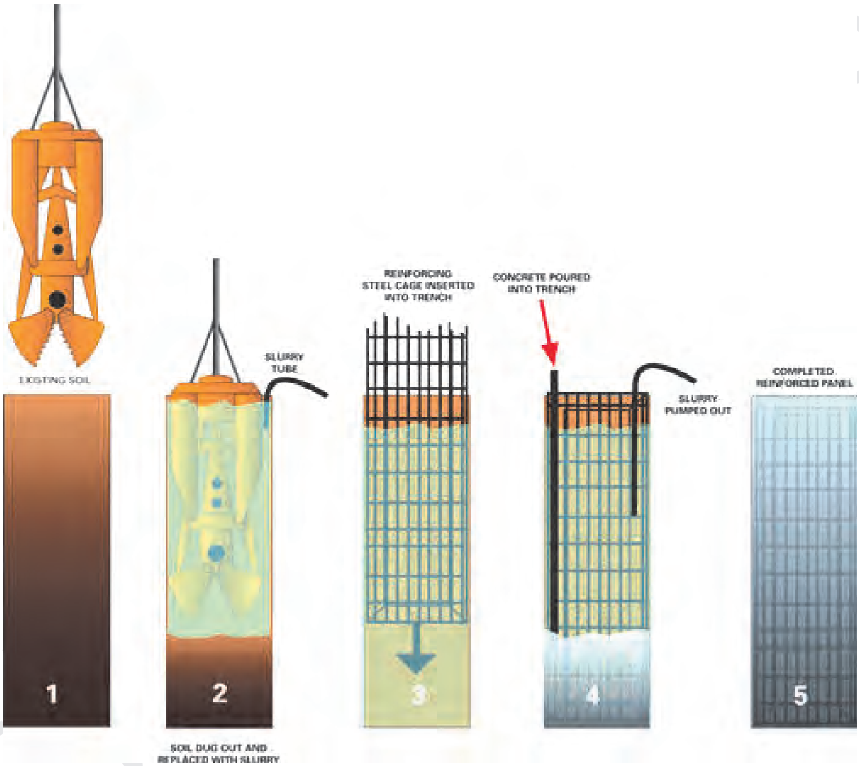
Underground construction is connected with much bigger risks than usual construction in the vicinity of adjacent buildings. To make the total risks acceptable it is required to address each risk factor separately. If it is impossible to eliminate them, then one should at least minimize them.

Examples of pit failures in various cities.

In Chapter 5 we discussed how to properly design underground structures. Now let us consider underground construction technologies.

There are two geotechnologies that have altered everything, namely, diaphragm wall and jet grouting. They gave a possibility to construct underground structures in places where it was perceived impossible in the middle of the 20th century.

Diaphragm wall and jet-grouting are the technologies which introduce possibilities of underground construction in the soft soil conditions.



The sequence of diaphragm wall construction in one bay.

Diaphragm wall is a monolithic reinforced concrete wall built in a very deep trench. The trench is dug by a special grab under protection of bentonite slurry. Fresh slurry is being added all the time into the trench on top of which a special reinforce concrete collar, called *fore-shaft*, is built. When a designed level is reached the grab thoroughly cleans the bottom. A reinforcement cage is put into the trench after which it is concreted with the use of vertically moving tube. Thus one bay (or "panel") of the diaphragm wall is formed. Its width can vary from 40 cm up to 1.5 m and its length in plan depends on the adjacency to neighbouring houses. The closer the adjacent buildings the shorter the bay. The bays are separated from each other by stopends with waterstoppers.

The depth of a diaphragm wall depends on soil conditions, dimensions of the underground construction and is determined by calculations (together with measures to safeguard it against horizontal displacements).

We had a pleasure to observe the work of Franki company carried out in Damrak – the main street of Amsterdam. An underground station was being built under the street. The tunnel itself was built by shield TBM method, that is a covered method (just like the way we do it here), and the station was constructed by open-cut technology, that is to say from the ground surface.

To construct a three storied underground structure which includes an underground station and commercial floors, along its perimeter a diaphragm wall was built with the width of 1.2 m and the depth of 48 m. The grabs were working just 3 m away from existing historic buildings, a strict monitoring of which was carried out. For the excavation they selected a top-down method under the protection of floor discs (however, the direction of the construction was only down, there was no need to go up).

If the same construction had been undertaken in St. Petersburg, half of the city would have been closed. And in Amsterdam they just blocked half the street. Traffic was jollying along the second half. The construction site itself was so narrow that they had to put a bentonite plant (where the solution is prepared and rejuvenated, cleaned from soil sludge) only in half a kilometre. Neat pipes filled with bentonite were laid along the embankment past the equestrian statue of Queen Christina. One should probably need to be born in a very small country to learn to work so finely. That is not how we do it! We would rather close an avenue for years so that nobody would bother us while we repave it with tarmac (it does not matter that you cannot see even one worker as far as your eye can reach, but what of the scale of things! Eh?).

Diaphragm wall – a champion of stiffness.

Diaphragm wall technology has some incontestable advantages over many other technologies of deep pit construction.

It is a champion of stiffness. The most sophisticated imported sheet piles are hardly equal to the diaphragm wall only half a meter wide. And stiffness is very important: the more flexible the retaining wall, the higher the settlements of the neighbouring buildings.

A diaphragm wall can be made extremely stiff owing to buttresses which are produced by the same technology.

As we have already informed you, Dear Reader, the stiffness of the diaphragm wall 1 m wide with 3 m buttress is equal to the stiffness of the flat 2.5 m wide wall! And this is something for fulfilling the needs of underground construction in complicated soil conditions of St. Petersburg.

Another very important advantage of this technology is the safety of the neighbouring buildings.

Prior to coming to this conclusion, in cooperation with *Geoizol* and *Franki* companies we carried out excessive field tests of this technology. The first successful trial in St. Petersburg was carried out on Komendantskaya Square. A 18 m deep 75 m diameter pit was supposed to be built there. Nobody in St. Petersburg had built such underground structures next to existing buildings before. But our teachers from *Franki* were scared by the behaviour of St. Petersburg soft soils: they had not come across such heavy but fluid soils before.

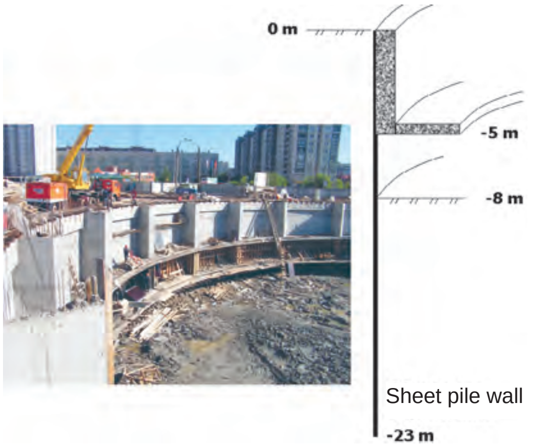
Therefore the technology was altered: cement was added to bentonite to increase the density of the solution up to 1.5 tons per m^3 (instead of 1.05-1.15 t/m^3 of pure bentonite solution). The walls of a borehole in difficult soil conditions are more easily held with heavier slurry. However, there is a catch: it is more difficult to supplant this solution with concrete, of which the density is 2.2 t/m^3. Therefore a decision was made to insert metal sheet piles into the trench made according to the diaphragm wall technology. We nick-named the resulting structure "sheet piles in sour cream". In the diaphragm wall constructed along a round pit only one small bay was classical monolithic diaphragm wall. We wanted to look how it would work in our soil conditions. (And these are the elements of scientific approach to real practice of the underground construction). While excavating we became certain that there are reasons for hope.

First trial to construct diaphragm wall according to modern technology (Franki, Geoizol, Komendantskaya Square).

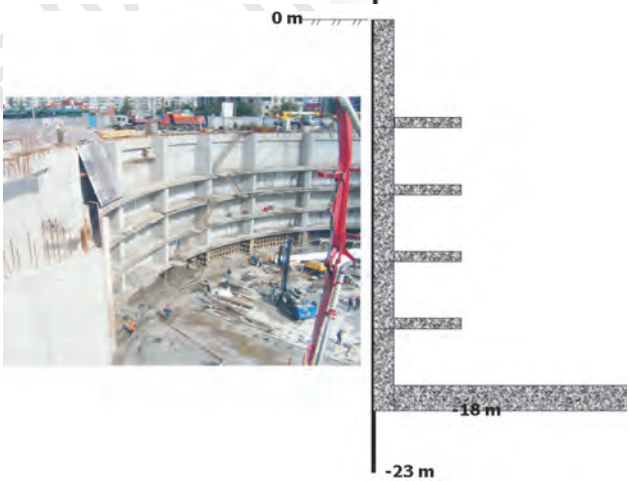
The second stage of the technology adjustment was the construction of the parts of the Orloff Tunnel's ramp on the left bank of the

Neva River. This area used to be called “The Sands”. Indeed, down to the level of 20 m there are sand sediments, which is not typical for St. Petersburg. However, it is very typical for other cities, where the diaphragm wall technology had proved reliable. Therefore, it was very important that local contractors got the hang of it just in this standard environment. *Geoizol* in cooperation with the specialists from *Franki* successfully produced the diaphragm wall, thus providing impermeability of the pits. The calculations and the design of this important structure was carried out by “Georeconstruction” Institute.

Detail of the Orloff Tunnel ramp – the first underground structure in St. Petersburg built by successfully applied diaphragm wall technology.



Construction of the first underground structure (2007): diameter – 75 m, depth – 18 m (the works were carried out by Franki and Geoizol, the calculations were done by "Georeconstruction" Institute). The cylinder is an ideal shape for an underground structure. Required stiffness is provided by the system of stiffening ribs.



Now we knew that the diaphragm wall technology had been mastered by St. Petersburg geotechnical engineers. However, both successful examples had been carried out in the "green field". Would the trench be stable under the protection of light bentonite slurry if a heavy building stood on one of its sides? No one had the answer. Further research was required (and this is science again).

Some of our colleagues including pure academics start being nervous for some strange reason when it comes to scientific research. The pro-rector of one respectable institution liked to clamour at various high gatherings: "Enough with the research. If we had been researching soils all the time, we would still not have built anything!" Well, being a representative of high science he certainly knew better. However, the world experience evidences to the contrary. No way can we, real practitioners, make designs without soil research. We managed to persuade an experienced developer, "Vozrozhdenie Peterburga" company, likewise. In the courtyard of an empty house that was to be demolished (in Zoologichesky Lane) just half a meter away from the façade wall a test pit was built, surrounded by a 30 m deep diaphragm wall of straight and T-shaped sections. The experiment was carried out with spectacular results. Despite all imaginable hindrances (works were carried out adjoining the building, at the depth of 20 m there were boulders, it was necessary first to crush them and then to extract them to the surface), the settlement of the house during the period of works totalled only 16 mm. The pit excavation down to 110 m evidenced that the resulted diaphragm wall was of a high quality both in the plane and, more importantly, in the T-shaped sections.

Zoologichesky Lane. First trial of diaphragm wall construction in adjacency to existing buildings (the experiment was carried out on the buildings subject to demolition).

Here, in Zoologichesky Lane for the first time in St. Petersburg a complicated T-shaped diaphragm wall was successfully constructed. This gives a lot of opportunities for underground construction in St. Petersburg (the works were carried out by Geoizol, design, calculations and monitoring were done by "Georeconstruction" Institute).

Now, after training on guinea pigs it is possible to move to a real project in the adjacency to existing buildings. We keep wondering how some designers without any shade of doubt take highly dubious decisions on underground structures. And like the devil from holy water they run away from the sheer thought to check their mad ideas at the trial pit. "What for?" – they ask. "We will test them on real buildings." Unfortunately, it is not a joke. In due course, the names and addresses of these "heroes" will become well known.

As for us, we were never inclined to conduct experiments on real residents. We prefer to carry out thorough field tests on trial pits rather than to start working in adjacency to existing building using a new technology, no matter how prestigious and foreign.

The results of our research were widely published and are available to all who are interested in geotechnical engineering. From 1998 we have been publishing scientific journal "Urban Development and Geotechnical Construction". Our research helps to avoid errors. To analysts it will reveal the real soil behaviour, to the designers it will help to find correct solutions, to the contractors it will show what geotechnology approbation in St. Petersburg is and how to adopt a technology to suit St. Petersburg soil conditions.

We should not forget that high technologies require high intellect. It is necessary to follow the standard procedure, technological regimes, *etc.* Otherwise, even the most advanced and reliable technology can be compromised. We will not hide the fact that once it happened even to the diaphragm wall, which, it would seem, had passed all trials in St. Petersburg successfully.

One contractor at first was not much concerned that with each new bay of the diaphragm wall over-consumption of concrete was gradually increasing, at the end more than two-fold (it must have been the client who was paying for the concrete). However, at one moment it occurred that it was impossible to excavate down to the design level of 24 m. The grab took the soil from the level of 9 m, put the soil to the dump, took another batch, but could not go lower than 9 m! They tried again, and again but the trench was only 9 m deep. They had been working in this manner till nightfall, when they gave up on such miracles and started pouring concrete into the trench only 9 m deep. They went on pouring and pouring but the bay would not fill. With great difficulty the contractor filled insatiable bay with concrete. And on the third day the Contractor started digging the next bay. They reached the level of 9 m and could not go deeper – below there was the concrete from the previous bay. And so on in Daniil Kharms[1] style. So the Contractor started to think and came to the

[1] Russian humorous writer

conclusion: "The culprit, – said he, – is an underground river with jelly banks, where the excavation under bentonite protection is impossible." Here in St. Petersburg it is always like that: should a building collapse or cracks form the same underground rivers are to blame.

However, if we moved from a fairy tale to reality it turns out that it is required to follow the technology, to maintain the required bentonite level, thoroughly clear it from soil and follow other boring rules…

But still, currently if you want to build an underground structure next to the existing buildings there is no alternative to the diaphragm wall technology. Both sheet pile and bored pile walls have incomparably lower stiffness. Besides, pile walls leak like a sieve. It is necessary to waterproof all joints which totally eliminates imaginary saving which, it would seem, should give investment advantage to a pile wall over a diaphragm wall. Besides, when a wall of bored piles with a significant diameter (600 mm and over) is produced there is no escape from settlements of adjacent buildings. They are caused by the same uncontrolled overdrilling which we discussed in the previous chapter. And the overdrilling (or over-excavation), as we know, directly depends on the degree of soil remoulding. The thicker the pile – the more pronounced is this effect. It is especially apparent when a row of secant piles is made.

If a pile is raw the drilling process may damage its body and if it is set than there is no escape from dynamic impact which turns soft soil into jelly which is easily and abundantly extracted to the surface. In this way three houses next to Nevsky Palace hotel were destroyed, as well as No. 6 on Michurinskaya Str., Nos. 26 and 30 on Ligovsky Pr., with No. 26 on Liteiny Pr. (Muruzi House) suffering some damage, *etc.*

Sheet pile wall can be quite efficient for relatively shallow pits (6-8 m). Today local contractors are equipped with a marvellous technology of pile pushing, which in time will doubtlessly oust the method of vibratory pile driving.

When pushed, sheet piles come into the ground like a knife into butter. The effect of displacement and overcompaction of soil (the negative effect which we mentioned when we were discussing pushed piles) is minimal in case of sheet piles.

For any retaining structures the task of providing their stability is quite an urgent matter. It is especially so in the adjacency to existing buildings. Let us remind ourselves of a regularity: *the value of the retaining wall displacement inside the pit is equal to the value of settlements of an existing building if it is situated adjacent to the pit.* It is impossible to eliminate the settlement by merely increasing the capacity of the retaining wall. Even a powerful diaphragm wall with buttresses has some difficulties with stability in St. Petersburg soil unless it is propped with horizontal struts. It is impossible to use anchors within the city boundaries: one might get into somebody else's territory or even under an adjacent building. It is quite an absurd situation: to save the adjacent building we try to get the anchor into the soil under the same building. Indeed, it reminds of Baron Munchausen, who gets himself together with his horse out of the swamp by pulling himself by his own hair.

So the choice is rather limited: either to build temporary metal props, or to cast reinforced concrete disc slabs which will later act as floorings (top-down method). In China even temporary props are made of concrete. The soil has to be excavated from between these props or through openings in reinforced concrete floors.

When the next level is reached the new props are installed or the new floor is cast. And so on till the bottom of the excavation pit. Props and floorings should be installed every 3-4 m. If they are built less frequently there is a danger that the retaining structure will develop large displacements. This task, honestly speaking, is not a pleasant one, especially in St. Petersburg soft soils.

Temporary reinforced concrete props on an underground construction site in Shanghai.

One has to work in constrained conditions: as if it were not enough that it is impossible to use usual machinery under the props or floors, pillar-piles that support the propping system also hinder the works. We will tell about the first example of top-down technology application in St. Petersburg in Chapter 22. But we would not recommend to use this method without urgent necessity even to a bitter opponent. It is not easy to excavate soft clay even from open pit. It sticks to the working machinery, equipment sinks in it, *etc.* On Komendantskaya Square (where thanks to the round shape it was possible to avoid top-down method and work in the open pit) excavator had to move on a pontoon made of 800 mm metal tubes so that it would not sink. First they excavated in one place, then they moved the tubes to another place and continued to dig.

To work under an already existing floor-slab in our soils is pure heroism. Soft soils tend to engulf even light-duty machinery.

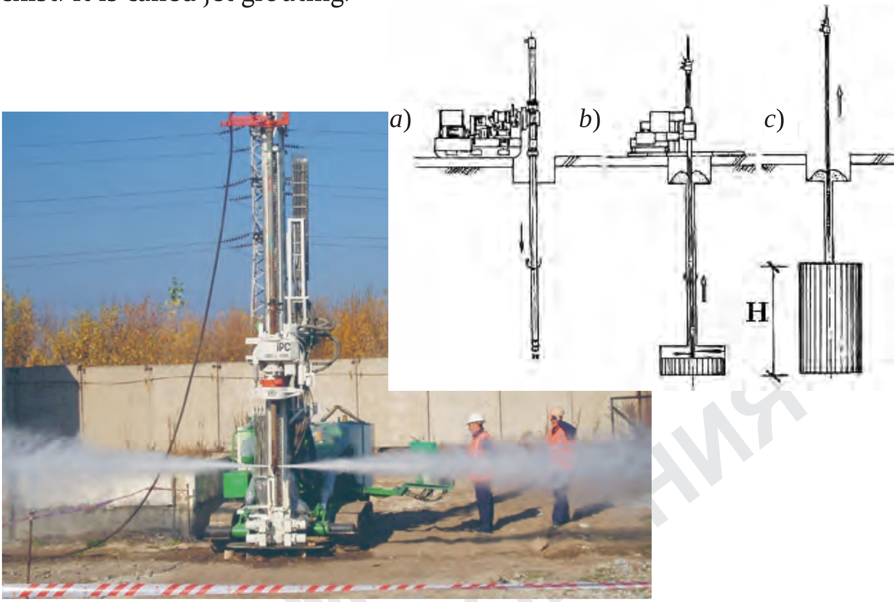
We have already mentioned that the best way to avoid top-down method of excavation is to construct a round pit. The ring supports itself. No struts are required. At most it might be needed to install rib-hoops on the levels of the floors. However, such shape is rare for pits. More often the shape of the pit repeats the shape of the site. But even in this case a philanthropic designer can contrive not to put men underground before their time. We have, for example, developed a geotechnical concept of big pits excavation (with the dimensions exceeding 100×50 m) where we managed to reasonably minimize the scope of works carried out in constrained conditions. An inquisitive reader can learn about it in Chapter 18, which tells about Mariinsky Theatre-2 project.

The value of the pit displacement is equal to the adjacent building settlements.

The gist of the concept is quite simple: along the perimeter of the pit two rows of retaining structures are built. The inner row is situated 10-15 m from the outer. Between the rows struts are built and 3-4 m of soil is excavated. Then the new struts are placed and 3-4 m of soil is excavated. This cycle is repeated till the designed level of the bottom is reached (supposing, 12 m). On this level monolithic slab is cast, after which a box reinforced concrete structure is built.

In plan it forms a closed frame (stiff contour) which is capable of withholding lateral pressure of the soil. If the site is elongated, then long sides are divided by crosspieces. Within the stiff contour the main scope of soil is situated, which can be excavated by the common open-cut method. Both retaining rows in this concept can be constructed of sheet piles (perhaps, it is only here that it can really compete with the diaphragm wall).

St. Petersburg soils have another insidious property. The pit retaining structures in such soils tend to form biggest displacement always below the current level of excavation. How to stop the displacement in places which we have not reached yet? How to put in a strut before the excavation of the pit? Well, in fact such method does exist. It is called jet grouting.



Jet grouting technology: on the photo – testing of the jet rig; on the diagram: hole drilling (a); mixing soil into pulp which is replaced by cementitious grout and pushed onto the surface level (b, c).

A rotating monitor is lowered into a borehole and through its nozzles a very high pressure (300 atmospheres) fluid jet of cementitious grout is expelled into the soil – hence the name of the technology. A sharp jet can cut metal, to say nothing of soil. The jet turns the soil into a pulp, which is pushed up to the surface. At the base a cylinder of mixed soil-cement grout is left. From these cylindrical columns constructed next to each other it is possible to create a body which surely cannot be considered to be concrete but which is two times stiffer than the original soil. Soil-cement grout mixture if properly prepared by the jet grouting technology works quite well as a strut which is formed in the soil even before the pit excavation.

Retaining structures in St. Petersburg soils form the biggest displacement below the level of excavation. This displacement can only be stopped by jet grouting.

Prior to recommending to use this technology in St. Petersburg soil conditions, we carried out its trial during the reconstruction of the house on Karpovka Embankment. It was planned to add two stories to a three- storeyed building and therefore it was required to increase the bearing capacity of its foundation. We suggested to make an area of strengthened soil under the foundation footing using jet grouting. The works were carried out quite successfully: the building did not suffer any noticeable deformations in the process, and after completion of construction works on additional stories its settlements totalled only 10 mm.

But most importantly, having extracted the core samples we got evidence that the stiffness of the strengthened soil satisfied the design requirements.

For the purpose of underground construction jet grouting technique was first used in St. Petersburg during the construction of the Second House of Mariinsky Theatre. According to our design 2 m of soil below the pit bottom within the boundaries of "stiff contour" should have been strengthened by jet grouting thus eliminating displacements of retaining structures below the excavation level and ensuring the safety of the neighbouring buildings and services.

Jet grouting technology allows to create not only horizontal cut-off screens below the pit bottom, but also vertical ones. But it is necessary to always remember that soil-cement mixture is not concrete. It is hundreds of times better than untreated soil, but hundreds of times worse than concrete. Soil-cement does not form monolithic mass, it consists of compactly closed cylindrical poles. Standards for reinforced concrete cannot be applicable to it.

So, jet grouting technology is undoubtedly a good thing, but do not ask for the impossible.

All technologies of underground construction have limits of their efficiency. The Geotechnical Engineer determines these limits with the help of three things: modern calculations, approbation and adaptation of the technology on a trial pit and geotechnical monitoring.

Chapter 16 – explaining geotechnical monitoring

The notion of geotechnical monitoring was introduced into Russian practice by professor Ulitsky already in the 1980s. Monitoring was included into codes for foundation construction (TCH 50-302-2004) as an obligatory element of supervision of construction in urban conditions.

No need in monitoring?

So, what is geotechnical monitoring: unnecessary additional expenditure or something useful for an investor?

First and foremost, monitoring is carried out to ensure safety of the existing buildings surrounding the site. It is necessary to constantly check their "health". Why? Because any new development within city boundaries is an operation on its body. The more complicated a construction project is (the closer it is to the excising buildings, the deeper it gets under the ground, the higher it rises), the more dangerous it is. Such construction projects are like an operation on the city's heart. They are inconceivable without constant monitoring over the health of the neighbouring buildings.

It is not rare that we see that simple observation is substituted for monitoring. They say it is like taking a patient's temperature. Now it is normal – everything is fine, now it started to rise – well, he must have fallen ill.

Such understanding of monitoring is no less than discrediting the notion itself. Why do we need a monitoring department when everything it is capable of is just stating: "Oops, there, look, settlement! Oops, settlement, again". And in some time: "Wow, there was a building there yesterday!!!"

It is a doctor who should observe the patient's health conditions, not a nurse, nor even a senior nursing officer. Only a doctor can explain the reasons why the temperature rose, how dangerous it is and how one should treat the patient. By the way, besides the temperature there are other parameters which are no less important.

To watch over the condition of buildings one needs surveyors capable of measuring settlements and tilts, a specialist in instrumental measurements capable of monitoring vibrations, cracks opening and forces in structures. But most importantly one needs a geotechnical engineer capable of analyzing the data on measurements, comparing it with the predictions submitted as part of the Geotechnical Substantiation, and drawing a conclusion whether everything goes according to plan or urgent change of technology or design solutions are required.

In other words, monitoring department can work effectively only if it is supported by a powerful geotechnical design group. Monitoring isolated from geotechnical engineering is not only inefficient, sometimes it is just dangerous.

The following situation is not a rare one. Permissible settlement tolerance for adjacent buildings (2-3 cm) is "used up" already during the period of piles or cofferdam construction, when excavation works have not yet started, not mentioning the superstructure. But the monitoring department is silent. This department, isolated from geotechnical engineers, is ignorant that these precious 2-3 cm should be spared for other more significant works, the settlements caused by which cannot be reduced to zero.

Monitoring is an instrument of risk management.

It is impossible to make a building weightless and retaining structures totally immovable. However, the impact of geotechnologies on neighbouring houses can be limited. It is possible not to allow use of dangerous technologies on the site. Surely, it is a responsibility of the designer, but a surveyor who carries out monitoring should not turn a blind eye to it. It is necessary to control sparing working regimes, and, needless to say, control over geotechnologies is one of the most important components of monitoring altogether.

Without analyzing the results of measurements, without comparing it with the geotechnical prediction, without control over geotechnologies and finally without diagnosing and choosing proper treatment, monitoring can turn into yet another means of administrative kickback.

When we insisted on including geotechnical monitoring into the codes we believed that it should become an instrument of risk management. And reducing risks for neighbouring buildings is in the interest of both the investor and the city.

Monitoring is not just measurements. It is calculated analysis and feedback to the design.

Monitoring not only helps to timely stop dangerous processes on the construction site. It also helps to protect the developer against unjust claims of the owners of neighbouring buildings, who usually are not at all glad because of your project. Monitoring will arm you with the data on true condition of the adjacent buildings and not just conjectures of ill-wishers. By the way, monitoring should start with condition surveys conducted on all neighbouring buildings, registering all existing defects. It is not just a requirement stipulated in the codes but the barest of necessities. Otherwise, everything that had happened to the house over its prior bicentenary existence will become your problem. Which can be expensive, indeed.

Part Four

Delving into the city's underground

Chapter 17, and in this part the only one, explaining advantages of underground development

Underground in art is something shocking the onlooker with its novelty. Underground in its direct (*i.e.* geotechnical) meaning is the underground space. For ancient cities it holds the history. But for such a young city as St. Petersburg it conceals the future. And here is why.

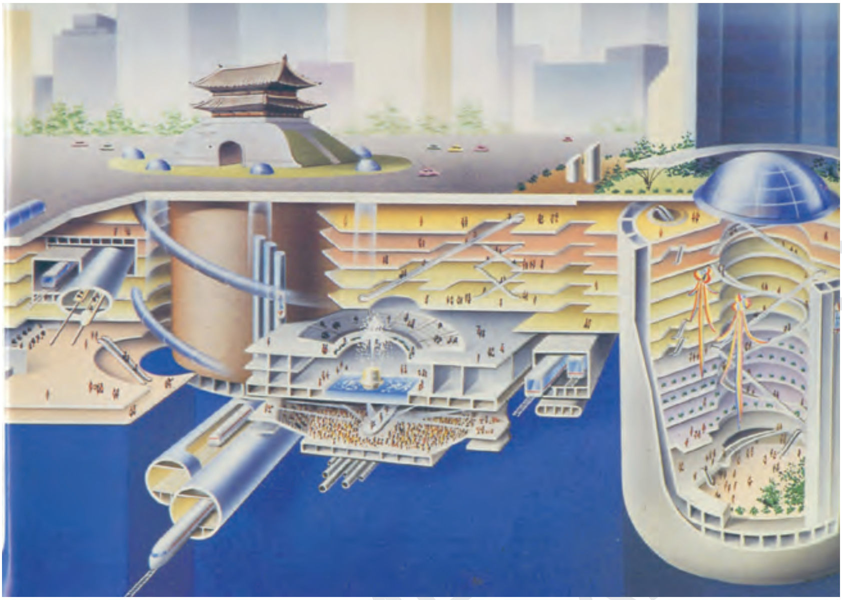
Any self-respecting state cares about preservation of its historic heritage – individual cultural monuments and architectural ensembles. The UNESCO protects multiple historic sites in all parts of the world, almost in every country. Our state also pays considerable attention to preservation and restoration of monuments. Some places protected by the international community are located in Russia. Among them – St. Petersburg in its historic borders of the erstwhile Russian imperial capital. One really can and must be proud of the fact that the shape of the city when it was the backdrop of the highest rise of the Russian culture has reached us unchanged, almost without any distortions or alterations. At the same time extreme challenges arise connected with the necessity to maintain safety of its historic centre. The matter is that historic St. Petersburg is a city which up to 1917 had been inhabited by 1,5 million people. For comparison's sake we would like to remind that in Venice, to which the Russia's northern capital is often compared, at the time of its blossoming there lived only 300 thousand people. It is obvious that to transform a city in which there used to live 1,5 million inhabitants into an open-air museum is a thing not only absolutely impracticable, but also harmful. Even in the small Venice turning the city into a museum proved to be a very complex process – the city-museum is being abandoned by inhabitants and a stamp of decline lies visibly on Venetian buildings.

Developing the underground space allows to preserve the historic city and make it convenient for life.

As is well known, an abandoned house quickly comes into disuse and desolation, the same can happen to a city which has become inconvenient for life. Therefore with a view of preserving the historic centre of St. Petersburg it is rather important to make it functional as a modern city, without distortion of its architectural outlook. This task contains two obvious extremes which apparently make it impossible to be solved. Here geotechnical engineering may come to rescue. Everything new, without which life in a modern megacity is inconceivable, should be hidden underground. Transport communications and traffic interchanges will be removed literally "off the face of the earth" providing convenient transport opportunities. There will be underground parking facilities relieving the streets of parked cars, which, like blood clots, are now clogging the city's arteries. All new functional areas and infrastructure necessary for the city will be placed into the subterranean domain. This way underground becomes a futuristic project.

This way of development has been chosen by many historic cities such as Paris, Madrid or Lisbon. In the two-million Madrid presently more than 50 transport motorway tunnels have been built, underneath the Puerta del Sol, in the heart of historic Madrid, lies a public transport hub, conveniently serving a network of underground metro lines.

It is possible to endlessly discuss the merits or otherwise of the well-known glass pyramid at the Louvre, designed by the architect I.M. Pei, but even its opponents recognize that the spacious underground facilities only enriched the historic building of one of the greatest world's museums.



Underground space of a megacity in Japan (presently the transport facilities of this futuristic project have been already constructed, the social and the business parts are being realized).

The underground vestibule of the Louvre in Paris, crowned with a glass pyramid.

Development of underground space is ascribed exclusively great value even in small historic cities of France. In one of them a two-tier parking facility has been arranged directly under a river bed, not having altered the habitual shape of quays at all. Thus one of the most essential problems of the city was solved – the historic streets were relieved of the ubiquitous cars, to the undivided pleasure of their owners, who finally received convenient parking places.

Soletanche-Bachy: construction of a two-storeyed parking lot below the river bed in a historic city in France; from left to right: construction of a waterproofing for the parking floor (the river water is temporarily running through a pipe); stacking of protective slabs – the artificial river bed; completion of construction, of which one is reminded only by the parking entry gates.

Today, in the times of global development of tourism, the greatest interest is turned to the cities which have kept their historic outlook – such as Prague, Bruges, or Venice. It is a secret that their absorption into the past is due in many respects to a rather prosaic reason: at some moment in their history (say, for one hundred years) these cities had become poor. A hundred years is quite enough for the inhabitants to start to appreciate the previous century's buildings as a historic relic. The realization that it is worthy of preservation starts to prevail over the desire to keep making infinite changes and to reconstruct absolutely everything. The same has happened to St. Petersburg: today it appears that not only separate edifices erected in baroque or classicist styles are worthy of preservation, but also the buildings until recently derided as *eclectic* (now commonly referred to as representing the style of *historicism*). Moreover, it was found out that it is necessary to also keep the so called background architecture. So, how is it possible to keep such an array of ancient buildings if they are, already by more than one hundred years, behind the modern concepts of convenience and comfort? Certainly, the temples should be given back to the believers, and palaces turned into museums and cultural centres. It is just the right time, it seems, to transform ancient theatres into museums as well, because most of them are hopelessly behind the contemporary development of theatrical technologies. For this reason in Europe the most complicated projects of reconstruction of the well-known theatres are being realized, such as the well-developed underground spaces under Milan's *La Scala*, Lisbon's *Teatro Circo*, and Moscow's *Bolshoy*.

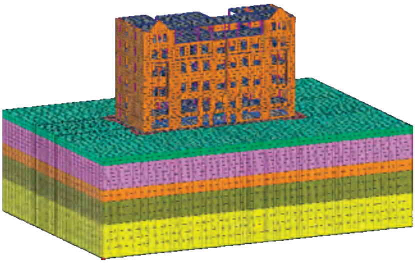
A.Pinto: construction of the second stage under the stalls of Teatro Circo in Lisbon.

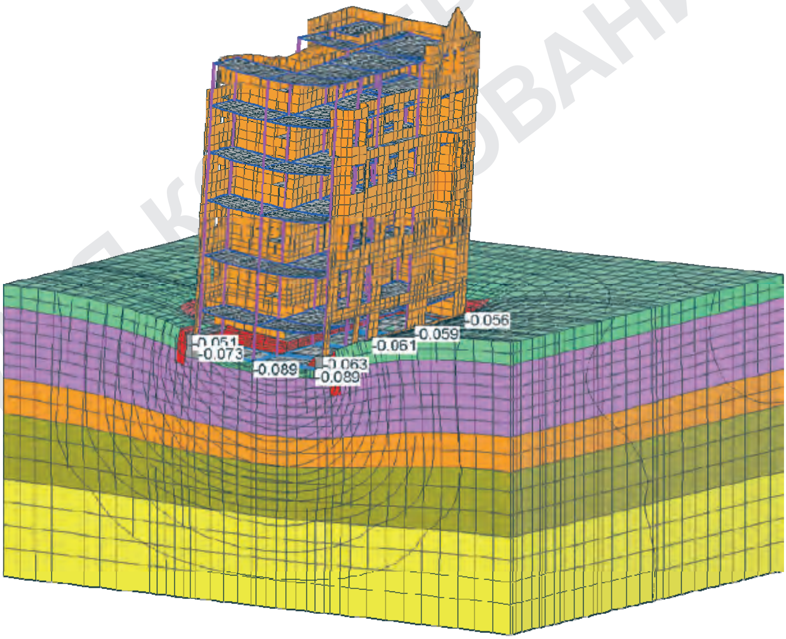
But then what should be done with the ordinary historic buildings of St. Petersburg forming its proverbial *view of stern and grace*?

Until recently this issue was resolved in the same way as in the rest of Europe: it is necessary to keep the buildings having valuable interiors, it is necessary to preserve all historic facades, and may be even all more or less older elevations forming the outlook of the streets, however, all internal extensions which used to house cart wrights' workshops, stables, servants' quarters and tenement apartments should be pulled down freeing space for construction of underground scopes and modern facilities. This arrangement was realized in reconstruction of several trading buildings of the XV century on the quay of the river Leie in the centre of the Flemish capital Ghent. Historic facades were carefully preserved and restored, but the shabby humdrum structures of the whole quarter were disassembled, with multilevel underground space, a comfortable hotel and restaurants put in their stead. The city has kept its historic outlook and acquired a new modern function of an attractive tourist centre.

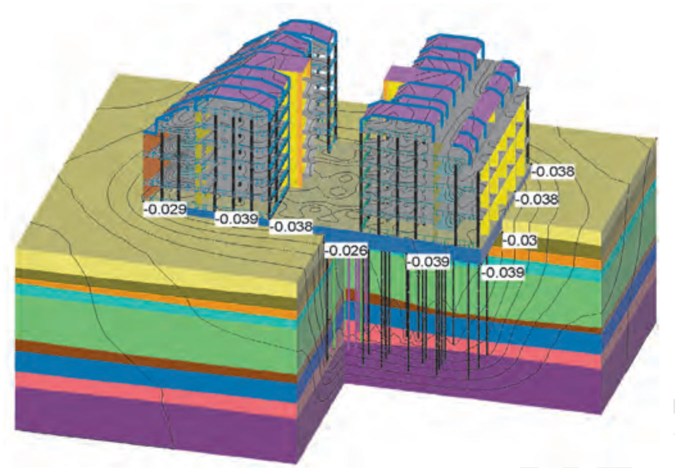
Reconstruction of historic buildings in Ghent (Flanders): the preserved historic facades conceal multilevel underground space and a comfortable hotel with restaurants.

In St. Petersburg the present writers developed a lot of reconstruction projects among which we shall name Malaya Morskaya 23, Bolshaya Morskaya 54, and Vladimirsky 19. These buildings, which kept their historic façades, became also convenient for the contemporary city dwellers.





The soil-structure calculation profile for Vladimirsky 19, St. Petersburg. External walls and the staircase were retained as having architectural value; critically dilapidated structures were replaced with reliable reinforced concrete skeleton on natural subgrade which the external walls were tied to; settlement values are indicated along the foundation.



Soil-structure interaction calculation profile for Bolshaya Morskaya 54 (original historic front walls were retained being underpinned with piles, the new structures were constructed on piled foundations); settlement values are indicated along the foundation.

Recently in St. Petersburg the noble desire to preserve the historic outlook of the city has begun to be expressed to the strongest excess. Its essence is simple: "*we shall stand in the way of absolutely any demolition or reconstruction*". In its core the idea has mistrust to construction specialists, incapable of providing safety of historic sites. It is necessary to mention the fact that first of all it is non-professionals who are to blame for compromising the idea of reconstructing historic buildings. Failure and demolition of ancient buildings which had received lethal deformations during construction of Nevsky Palace Hotel on Nevsky Avenue, and similar doleful fate of houses on Michurinskaya Street are still fresh in the minds of the populace. And now even the contemporary history gives us new examples of settlements of residential buildings reaching 100 mm (!) adjoining the trade Centre "Stockman" on Nevsky Avenue and the site of the Second House of Mariinsky Theatre (about those you can read in more detail in Part Five of this book). It must be said that the last examples are sometimes presented as successful projects of underground construction. It is difficult to agree with this: it is impossible to use the term “successful” in respect of a fivefold excess of permissible settlements the existing buildings received.

So, maybe, those people who stand up for a full prohibition of construction and reconstruction in the historic city centre are in the right? Maybe, but it must be borne in mind that should this approach win, the city will inevitability fall into desolation. It is impossible to transform a huge city with boundaries of the late 19-th century capital into an open-air museum. Hence, there is no alternative but to develop the city's underground space and reconstruct the morally outdated building quarters. This can only be done on a high professional level – both architectural, and geotechnical.

Examples of destruction caused to historic buildings during construction of underground volumes for “Nevsky Palace Hotel” on Nevsky Avenue (top left and right), and an apartment house on Michurinskaya Street.

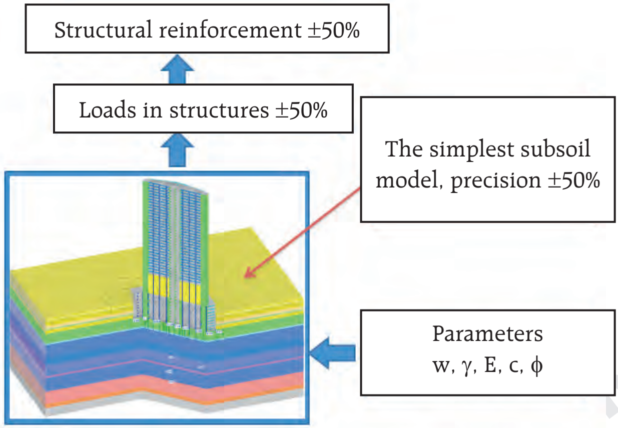
A good example for emulation – a multilevel underground space around a historic building in Lisbon the concept for which was developed by A. Pinto, a well-known Portuguese geotechnical engineer. The building was left standing on an island of soil framed by a diaphragm wall which was held together along its contour with prestressed cables. Around it a multilevel underground structure was constructed. Certainly, rather favourable site conditions (sandstone and limestone comprising the subsoil of the historic building) were conducive to the successful solution of this most complicated geotechnical problem. In St. Petersburg bedrock is located at the depth of more than 200 m and consequently has no practical value for construction.

A.Pinto: construction of an underground volume around a historic building in Portugal.

As our Dear Reader remembers, St. Petersburg was founded "*on banks of mosses and wet grass*" on a massive bulk of quaternary deposits of low and medium degrees of lithification, in simple terms called weak, or soft soils. The bearing stratum for historic buildings are lacustrine and marine sands located close to the surface. They act as an original natural sandy cushion for buildings whose height was limited by the eaves-level of the Winter Palace (24 m). Apart from the ethical meaning (it was an indecency to build a house higher than the imperial chambers), this rule had also a geotechnical consequence: it limited pressure from buildings onto subsoils. As a result the subsurface sands well coped with the role of the pillow redistributing pressure upon weak underlying subgrade strata – the lacustrine-glacial fluid loams.

On the pages of this book we already mentioned that these soils possess specific properties, ignorance of which can lead to dire consequences for any construction in the city centre, and for underground construction in particular. They are structurally-unstable soils. The machinery-related influences accompanying building activity, provoke infringement of structural bonds in these soils on account of which they lose properties of a firm body and acquire qualities of a liquid medium. It is necessary to say that this liquid is rather heavy, its density is equal approximately to 2 ton/m^3, *i.e.* it is twice heavier than water. The effect of infringement of structural bonds is expressed, first of all, in increase of mobility of the soil medium: its viscosity drops to minimum value and, consequently, the rate of shear strain development sharply increases. This phenomenon was studied by us on the whole range of trial sites where deep foundation pits were arranged. If in soil of natural composition lateral pressure value is usually half the vertical, at infringement of structural bonds it increases twofold, becoming equal to vertical pressure, as is regular for a liquid medium.

Unfortunately, numerical modelling programs available to designers today do not allow to adequately represent features of soft clay behaviour, being structurally unstable media for which it is characteristic to have viscoplastic type of deformation.



We do not advise geotechnical engineers to approach the structural designer with the accuracy of settlement calculations such as shown in this figure.

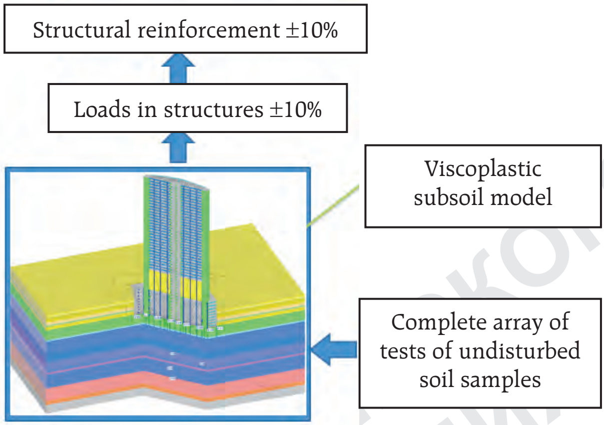
The majority of models of soil behaviour, realized in popular software complexes, either do not consider development of deformations in time at all, or have preconditions which are not true to real behaviour of soft soils. Moreover, even without taking into account the factor of time, applicability of these programs appears seriously questionable. Complexities arise even when we attempt to simulate by means of a numerical modelling program elementary laboratory experiments: odometer compression according to the open scheme (in conditions of free expulsion of water from the sample) or the triaxial test according to unconsolidated-undrained (*i.e.* closed) scheme. Thus it turns out that for weak soils the existing models are not always in the position to correctly describe both mentioned standard soil tests with the same set of parameters. How then is it possible to entrust to a program calculations of projects with complex loading geometry if they are not even true to modelling elementary tests?

As they say, “out of dire necessity”, the St. Petersburg geotechnical engineers had to engage into development of special models of soil behaviour. Based on twenty years' research, we developed our

viscoplastic soil model representing features of St. Petersburg clay behaviour under loading and unloading, incorporated into the "Library of Models" within our *FEM models* software package.

The significance of the viscoplastic model for computing underground structures can be explained as follows. During deep excavation unloading of subsoil takes place. Soil deformations in this case are not in any way connected to consolidation (as there are no compressive pressures). In popular programs development of deformations in time either is not considered at all, or is linked exclusively to consolidation, *i.e.* with water expulsion from soil under the action of compressive stress. How then is it possible for the designer to calculate deformation of a soil bulk during deep excavation by means of such programs when in reality no consolidation occurs, but it is necessary to account for the time required to install a system of struts? This problem is actually rather ordinary but absolutely inaccessible to existing software products. The matter is that in conditions of bulk excavation no expulsion of ground water takes place, whereas the prevailing deformation, developing over time, is that of form-change (if you were to trace any cube of soil inside the bulk you would see that it alters its shape and moves along, but its volume remains unchanged). These deformations, which completely define behaviour of soil during deep excavation, have for some reason been overlooked by software developers, in spite of being radically important even for superstructure construction.

The existing programs for prediction of deformations development can offer only the so-called theory of seepage consolidation which considers precisely the process of water expulsion from soil which is absolutely irrelevant for designing deep excavations. Certainly, it is possible to find programs that take into account the effects of creep (soil "crawls" under loading without changing its volume, just like a caterpillar). But their model is constructed in a peculiar way: deformations of shear are calculated through deformations of consolidation, physical validity of which is rather dubious. (If one admits existence of such connection it will appear that water can flow only depending on whether we encourage it by squeezing it or not).



The viscoplastic model provides necessary accuracy of soil-structure calculations

Thus, introduction of the viscoplastic soil model realized in our *FEM models* software and making it available to geotechnical engineers makes it possible to design deep excavation pits with account of the time it takes to construct them, as well of their influence on surrounding historic architecture. Owing to the viscoplastic soil model, being the tool of reliable prediction of soil behaviour, we get a prospect of scientifically substantiated development of underground space, excluding the threat of danger to surrounding historic buildings. We subjected the viscoplastic model to the most rigorous verification for its conformity to *in situ* test results on a series of test pits in St. Petersburg, as well as to standard laboratory and field soil tests (a number of publications in various scientific editions is dedicated to this subject).

However, it would not be out of place to reiterate that application of effective numerical calculation software does not relieve the analyst of the obligation to verify his results by means of traditional engineering methods. It is necessary to bear in mind that the traditional methods are more than two centuries old already and the mankind has gained significant experience of their application, whereas numerical methods have been used in daily design practice for only about a decade. As you, Dear Reader, have already had the opportunity to see in Part Two of this "Guidebook", if the problem is stated correctly and the theoretical basis is the same, the results of calculations according to the engineering and the numerical methods coincide. The advantage of numerical methods is not at all a more favourable design solution, but the opportunity they afford of considering complex geometry of loads and various stages in construction of underground structures. Therefore during design it is necessary to always solve the whole series of numerical problems, researching reaction of soil and existing buildings to various technogenic factors, also recalculating each of the problems by means of traditional analytical methods. The viscoplastic model is a rather convenient tool for this purpose.

Let us not forget that underground construction in city conditions on soft soils is an activity associated with increased risks. And, hence, we believe, design of deep foundation cofferdams necessarily entails the same degree of attention and care, as construction of other structures of high level of responsibility, because accidents on excavation pits can lead to human casualties. For such structures it is required to carry out calculations to counteract the so-called *progressive collapse*. For the case of a deep foundation pit it means not only consideration of failure of a single element of design, but, mainly, studying the potentials for onset of *force majeure* technological situations.

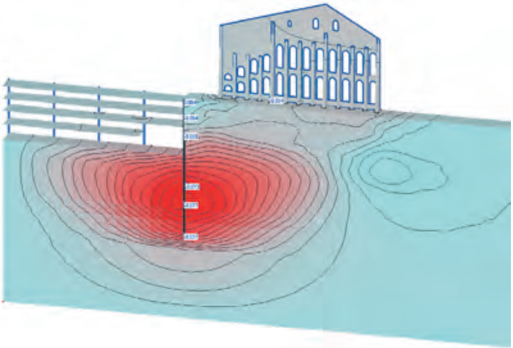
Structural instability of St. Petersburg clay imposes on us a choice of way which we need to traverse when designing an underground structure.

The first choice consists in making the most of properties of undisturbed soils. In this case expenses on shoring and retaining structures appear minimal. But the risk of losing natural structural bonds in soil due to infringement of technological regimes and technogenic influences on and around the site remains rather high. Thereat the deadlines of works inherent in the project can also be infringed. All these risks are quite real (especially in Russian conditions) and can lead to negative consequences down to destruction of the existing buildings.

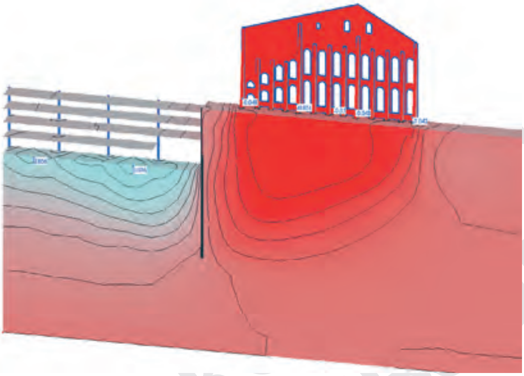
The other choice presupposes inevitability of infringement of natural soil structure or a possibility of uncertain delays of pit construction deadlines. In this case the soil will work as "a thick liquid", and the actions providing permissible deformations of the existing buildings appear to be beyond economic feasibility. Obviously, both the first, and the second choice in themselves lead us into a cul-de-sac: the first does not provide safety of the existing buildings, the second rules out any prospects of underground construction. Is there a way out of this deadlock?

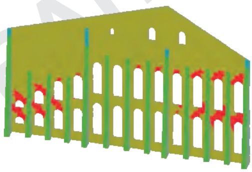
In our opinion, no search of a third way here is required, it would be enough to coordinate the first and the second way in order to solve the problem. The obvious reason of the inherent contradiction is that the "optimistic" calculations (assuming preservation of natural soil structure) and the "pessimistic" predictions (assuming full infringement of soil structure) cannot share the same criterion of permissible deformations of the existing buildings.

We suggested to introduce the following requirement to design of deep foundation pits: ***calculations of underground structures in congested city conditions should be carried out based on two groups of limit states: not only for the designed structure (which is routine), but also for the existing buildings.***



When calculation is performed for the second group, designs of the cofferdam and its shoring are selected in such a way so as to provide permissible deformations of the existing buildings. When calculation is performed for the first group, ultimate permissible settlements of buildings are defined so as to correspond to ultimate possible loads in their structures. Parameters for the cofferdam should rule out the threat of collapse of the existing buildings.





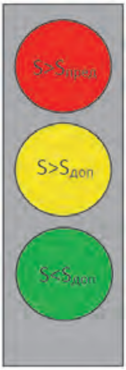
Calculations for the second group of limit states (for deformations) should be carried out based on usual requirements for permissible additional deformations of existing buildings that may be brought about by the sum of all possible influences related to construction of the project. On the basis of this calculation the design of the cofferdam and the system of its shoring (struts and props, *etc*) is selected to provide permissible deformations of the existing buildings on condition that the approved sequence and rate of works, with observance of routine technological regimes, are maintained. In this case it is very important to precisely predict rate of strain development in the subsoil, which is possible to achieve using the visco-plastic soil model. Here appears the necessity of precise coordination between the solution obtained by the geotechnical engineer (the analyst) and the schedule imposed on the contractor concerning the maximum permissible timeframe of each stage of works, significant for safety of the existing buildings. This timeframe should be coordinated already at the stage of forming the geotechnical concept. It is obvious that the most economic solution for the retaining and shoring structures can be obtained if the natural bonds of soil remain undisturbed. To realize design solutions oriented at preservation of natural soil structure it is necessary:

1 – to limit technogenic influences during construction of the foundation pit (to ensure there will be no dynamic influences both inside and around the pit, to exclude piling works, driving sheet piles, to limit movement of traffic around the pit, *etc.*);

2 – to ensure no possibility for infringements of works sequence and sparing technological modes;

3 – strictly observe design timeframe for each stage of works on foundation pit construction.

Obviously, there are high risks that these conditions will not be observed both for objective and subjective reasons (a delay in the project financing, mistakes of builders, and so on). We are profoundly convinced that a design based exclusively on the assumption of safety of the natural soil structure, and lacking tools to withstand onset of emergency situations, has no right to exist. Here the analogy to the approach commonly exercised today in order to withstand progressive collapse would be quite pertinent: a mistake in implementation of works or infringement of their timeframe should not lead to rendering catastrophic damage to the existing buildings. For this purpose it is necessary to introduce the concept ***of calculating the existing buildings for the first group of limit states*** into the practice of geotechnical calculations. The existing buildings should be calculated for durability and stability in the face of influences rendered by the underground construction, connected with uncertain delays of works and infringement of natural soil structure. In other words, infringement of sparing technological modes and timeframe of works should not lead to destruction of the existing buildings.

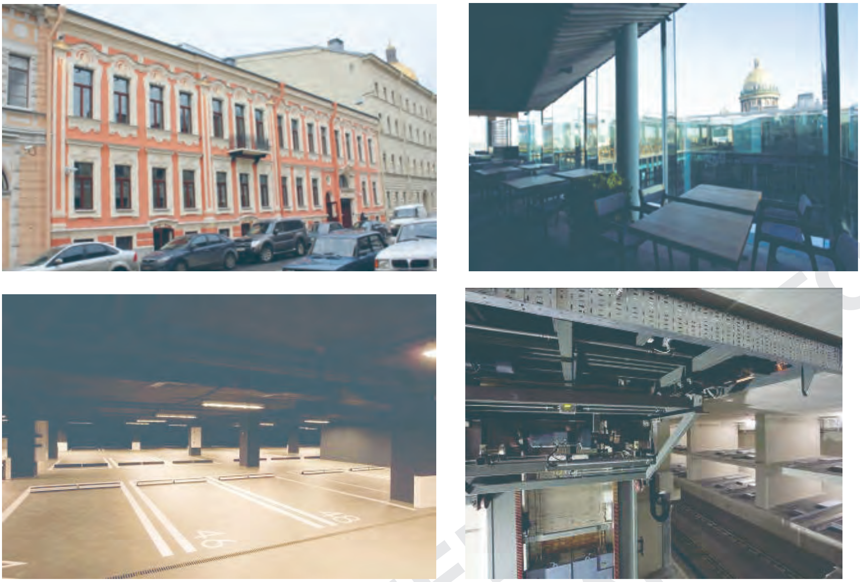


In practice of the leading geotechnical firms of the world such approach is not new; during interactive monitoring of complex challenging projects “the traffic lights principle” is realized. The green light denotes absence of problems, the amber is lit when deformations surpass the permissible values, and the red flashes once the ultimate maximum value of deformations has been exceeded.

The best testimony of the viscoplastic model’s correctness and efficacy of the suggested approach to designing underground structures based on two groups of limit states ***for*** the existing buildings are the projects designed and constructed in adherence thereto. Among them is the underground volume under the monument of cultural heritage called the Stone Island Theatre, listed as a UNESCO protected building (you can read about it in more detail in the final part of our book), as well as the three-tier underground parking facility arranged inside a historic quarter on Pochtamtskaya Street.

For a future business centre it was required to create an underground parking for 160 cars and a modern building in place of old sheds, cart wrights’ workshops and pre-revolutionary servants‘ quarters, having preserved historic buildings lining the street. Again, according to the fashionable contemporary “safeguard-everything” ideology it would have been necessary to wait until people willing to make direct use of those shabby ancillary structures have been found, *i.e.* those ready to use them for keeping fire wood, housing horses, and living in tiny rooms for serfs with 2,3 m ceiling height. It could have been possible also to wait for those structures to collapse by themselves, becoming *irreversibly damaged* (that one factor only, in the opinion of the “heritage guards”, makes one entitled to have the buildings disassembled). It seems that such “guarding” approach conceals in itself a delayed action explosive device: today it scares away investors, and tomorrow it will lead to dilapidation of the city. Fortunately, city authorities made the reasonable decision: everything new had to be concealed inside the project and remain invisible from any point on the ground surface.

Construction of the underground volume of the office block on Pochtamtskaya Street; design by "Georeconstruction" Institute. Top left: sheet pile driving under strict vibration level control; top right: construction of the rigid jet-grouted disk below the bottom of the parking "safe"; bottom left: basement floor excavation after soil strengthening actions underneath foundation footings of the existing buildings; bottom right: excavating the pit for the parking "safe" under protection of a short sheet pile wall and the disk of the jet-grouted soil.



The preserved building on Pochtamtskaya Street; the panoramic restaurant invisible from any point on the ground surface; the first underground level; the parking "safe".

Geotechnical engineers of "Georeconstruction" Institute took part in development of the project simultaneously with architects, which brought conceivably favourable results. Their mutual cooperation took shape in rejection of the original idea to arrange a two-level underground volume throughout the entire area of the courtyard cleared of all shabby structures. Instead of that a basement-level parking floor ("half-underground", as it were) was arranged (which allowed to limit strengthening of the adjoining buildings by the elementary underpinning actions in the contact area "foundation-soil") in which 100 cars could be placed, and in its middle, at a sufficiently remote distance from the historic structures, a double-tier parking "safe" capable of housing 60 more cars. Depth of the excavation for the parking "safe" totalled 8 m, whereas the cofferdam was constructed from short sheet piles with a horizontal disc of jet grouted soil at the bottom. As a result the underground volume proved quite profitable and simultaneously (which is far more important!) absolutely safe for the existing buildings. For the entire period of construction settlements of the existing buildings did not exceed 1 cm.

The example described above testifies, first and foremost, to the realistic possibility of safe underground development in the historic centre of St. Petersburg. The knowledge of laws of behaviour of weak soils under loading and unloading, including knowledge of changes in their rheological properties at infringement of structural bonds, allows to design underground structures and to choose appropriate technology for their construction, so as to exclude the risk of damaging historic buildings in adjacency. Secondly, this example testifies to the advantage which development of underground space can bring to a historic city, if for calculations and design one uses correct models representing the complex "underground structure/subsoil/historic building".

For the Dear Reader to dispense with any doubts regarding necessity of developing underground space, we shall try to dispel the fog with which some people often try to surround certain projects of underground construction. For this purpose we shall analyse efficiency of geotechnical solutions on some projects in St. Petersburg. We shall see that the settlements of the existing buildings exceeding 10 cm, are not at all inevitable, they can and should be avoided. For this purpose it is necessary to correctly predict soil behaviour and to adopt appropriate design solutions on the basis of adequate calculations.

Part Five

Rake Walking
(for adventure lovers)

A special new kind of sport has recently appeared. It is called rake-walking. We would not say it is an exclusively Russian sport. However, in the West everything is so interestingly arranged that one who steps on the rake just for once in his life keeps hearing humming in his head until the end of that very same life. We, the Russians, have our own specificity here, too: one steps on the rake, but the bump on the head appears on the head of someone completely other...

Chapter 18,
explaining the passions for "Mariinsky-2"

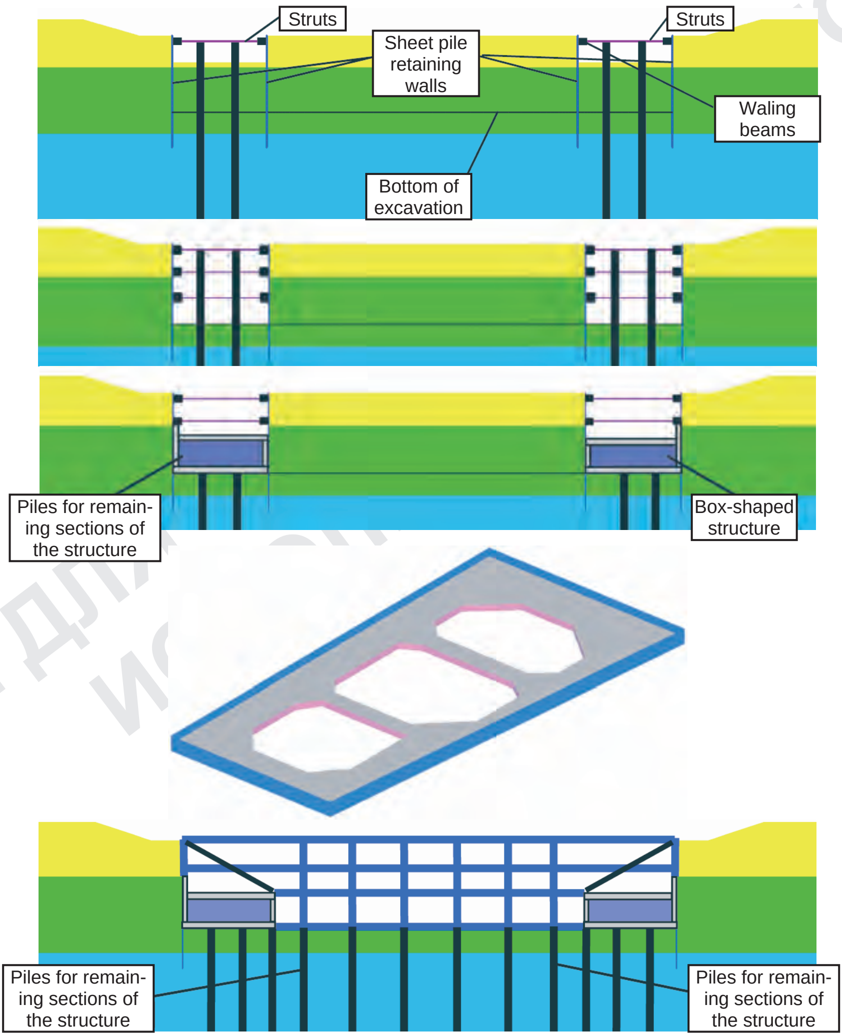
French magazine *Le Moniteur* had once undertaken a curious analysis of state-funded construction projects and discovered an important regularity: the increase in cost of construction is directly proportional to extension of its timeframe. Timeframe extension is helped by: a) saving money on site-investigation (the designer sits and waits until the new site-investigation is completed); b) unexpected discovery that the initial concept of construction is not good for anything and that several years of activity were spent in vain; c) sudden revelation, when the scheduled project completion is near at hand, that money has run out but no one has even thought about beginning work yet; d) something else. Besides, for some reason the state funded projects possess one amusing feature: they, more often than private construction undertakings, get their designers, contractors and even clients replaced. Each time their replacement is undertaken for the benefit of successful completion of construction, the cost of which as a result appears two or three times above the originally allocated amount. The nature of this phenomenon still remains a most profound mystery. It is known only that this mystery is always connected to "*all the fountains of the great deep breaking up*". It is regretful that every time there occurs underground space "de-*felon*-ment" at grand and large scale, the fact of such "de-*felon*-ment" not always prevent those fountains from breaking up.

In this context the history of construction of the Second House of Mariinsky Theatre looks so trite and so widely publicized in press that here we shall talk only about its geotechnical aspect.

Imagine that the year is 2005. It is necessary to design an 11 m deep foundation pit with plan dimensions 150 by 80 m. We, Dear Reader, do not yet have at our disposal the diaphragm wall technology. It is certainly known from its western and pilot Moscow applications, but in St. Petersburg there had been no positive experience yet. Foreign experts, having got acquainted with our soils, throw their hands in the air: "You can construct a diaphragm wall in your ground conditions only if you mix all your soil with cement first!". Secant pile wall is still a "bad egg" and a big sore in the public eye: the wounds of failures at Nevsky Palace Hotel, on Michurinskaya Street and the "burrow" near the Moscow Railway station have not been licked whole yet. As they say, here are your sheet piles, please spare no expense and enjoy yourself thoroughly! Otherwise, no approvals by the State Expert Board – you may experiment, but well away from signature projects funded by the state.

And there was yet another burden: according to the initial concept the theatre building is cut top to bottom with acoustic joints, and consequently the floor elevations above the two underground levels are variable here and there by about half a meter. There is neither a uniform level nor a continuous disk of intermediate floors which could be used for shoring sheet piles against horizontal displacement. But there are soft clays, in which no one has ever managed to construct a deep foundation pit without somehow damaging the existing buildings. And they are located just 15 meters away.

In this complicated geotechnical situation the experts form "Georeconstruction" Institute devised an interesting way of constructing the foundation pit which was cheap, simple and reliable. The idea was elementary: can we or can we not excavate a deep trench? The answer is "yes" if we protect it with, say, two lines of sheet piles and install shoring in between, while we excavate. Calculations showed that in order to preserve the natural structure of soft clay soils for 11,5-m deep trench three shoring levels would be enough. And can we make this trench 12-15 m wide? Well, why not, indeed? We excavate down to 3-4 m. Install a line of shoring. Then excavate another 3-4 m. And so we repeat until we reach the design level for the bottom of the foundation pit (down to 11 m). Then can we provide this trench along the entire contour of the underground structure, lined with sheet piles and shored appropriately? And then can we install a box-shaped reinforced concrete framework which is capable of sustaining lateral soil pressure when we are ready to excavate inside the contour? As the footprint of the building is elongated the longer sides of the frame need to be divided with transverse crosspieces. And Robert's your father's brother! Top-down method is not necessary, and neither are shoring disks of intermediate floors; two thirds of works can be conducted in the open foundation pit. Simple, cheap, safe and quick.

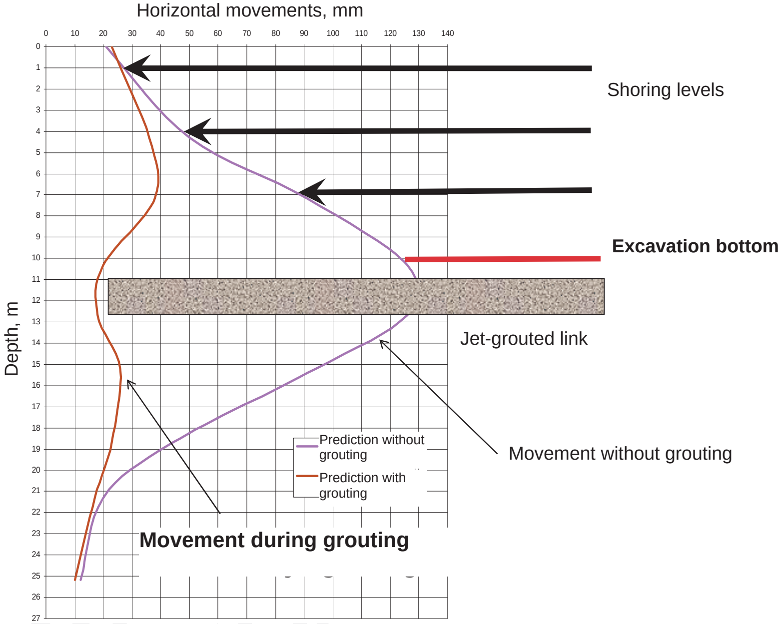


Sequence of excavation for the stiff reinforced concrete frame cofferdam solution.

The first deep foundation pit ever to be constructed in congested historic environment of St.-Petersburg required special treatment prescribed by the international construction standards – the interactive design. It must be said that that approach showed its efficiency when first difficulties appeared on the site. During construction of the pile field (bored piles had been designed as casing-protected) the contractor allowed overdrilling resulting from which the existing buildings developed settlements of up to 20…30 mm. Designers raised the alarm: what good is it, if the whole tolerance on permissible deformations for the existing buildings has been "used up" on the elementary and, obviously, tried and tested technology! That tolerance was so necessary for them at the following stage – the excavation of the foundation pit.

Overdrilling caused remoulding of soft clays. Soil with undisturbed structural bonds reacts to excavation inside sheet piles rather well. There is still time then (even if only one week) to install next line of shoring. But remoulded soil at once starts pushing on sheet piles, moving them inside the excavation. In this case, when excavating for the next level and installing another line of shoring, time is already counted by the hour. Should you be late, the sheet piles will have moved by several centimetres, with the existing buildings following in their wake.

Considering these circumstances, we suggested the following compensatory action: to install a strut at the depth of 12 m, where it was the most necessary, not after, but *before bulk excavation!* To realize this seemingly utopian idea there was yet untested technology of jet-grouting, now successfully practiced in Russia. Between the lines of sheet piles from the ground surface consistent boreholes are drilled and a layer of 2 m thick structurally fixed soil is formed below the bottom of the foundation pit by means of a jet rig. The stiff disc inside the soil thus formed is rather effective at the small distance between the retaining walls. Owing to it, flexibility of the walls of our trench sharply decreases, and the settlement increment on the existing buildings generated by the excavation is guaranteed to be limited to 10…16 mm. Added up to the settlements already accrued (20…30 mm) it still enables one to use only the most elementary making-good procedures on grouting the opened cracks during repair of the existing buildings and does not require expensive actions connected with strengthening the subsoil and foundations of the existing buildings.



Efficiency and layout of the jet grouted soil link.

This actually was the main principle behind the concept of "interactive design" – to take decisions in view of the changing geotechnical situation on site.

The described geotechnical concept was recognized as efficient by domestic and foreign experts whose opinion had been sought.

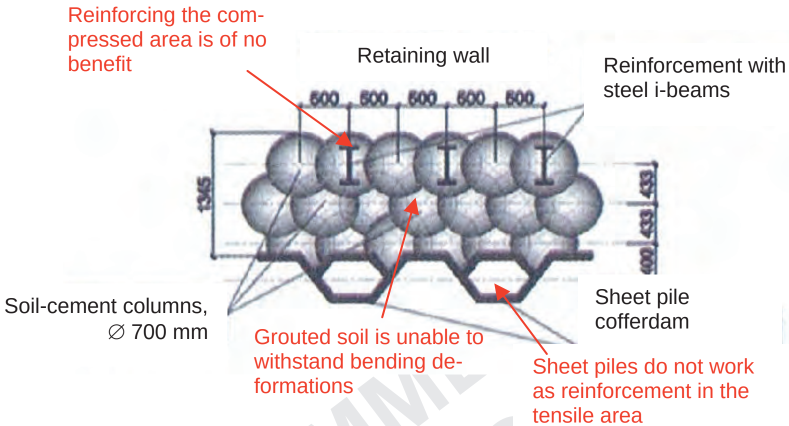
At that crucial moment, as, apparently, it must always happen under the laws of the genre, the working geotechnical concept was dismissed together with its authors. A bit later even the architectural concept itself followed in their wake (together with its author, again, the well-known French architect Dominique Perrault, the winner of the international architectural competition held in Russia for the first time in 70 years).

The alternative designer suggested a variant of constructing the underground space based on the top-down approach which was absolutely counter-indicative to Dominique Perrault's architecture.

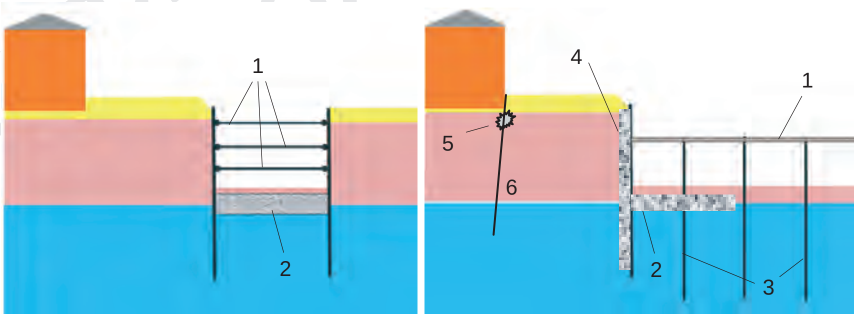
Whether the design by Dominique Perrault was too innovative for the outlook of St. Petersburg and the industrial box of the Canadians was traditional enough – let this be judged by experts, connoisseurs of architecture and St. Petersburgers not indifferent to what happens to their city.

In the foundation pit which was 12-meter deep only one (!) level of shoring links (at the depth of 4 m) was provided. The layer of the strengthened ground made under the bottom of the excavation along the contour of the foundation pit could not work anymore in the new adopted scheme. Indeed, what kind of strut is it – a bit on the one end, a bit on the other, and a hole in the middle!? It is surprising that this layer of grouted soil, absolutely useless under the new solution, attracted so much attention on the part of the press and the officials.

The only compensation for the abandoned levels of shoring struts in the new design was the construction of the strengthened bulk of soil 2 m wide, outside the external contour of sheet piles. The same jet grouting technology was used for that purpose at a grand scale. For some strange reason No. 40 i-beams were installed into the grouted soil on the outside, having 1 m spacing between them. There is no doubt that grouted soil is capable to work under compression. Therefore, reinforcing the grouted zone with i-beams was absolutely unnecessary. Grouted soil is not capable of withstanding bending deformations. The function of reinforcement in the area of tensile deformations was performed by sheet piles, to which, according to the opinion expressed by the authors of the new design, grouted soil should have stuck excessively. The sheet piles did not know about that and remained, as before, simply sheet piles.

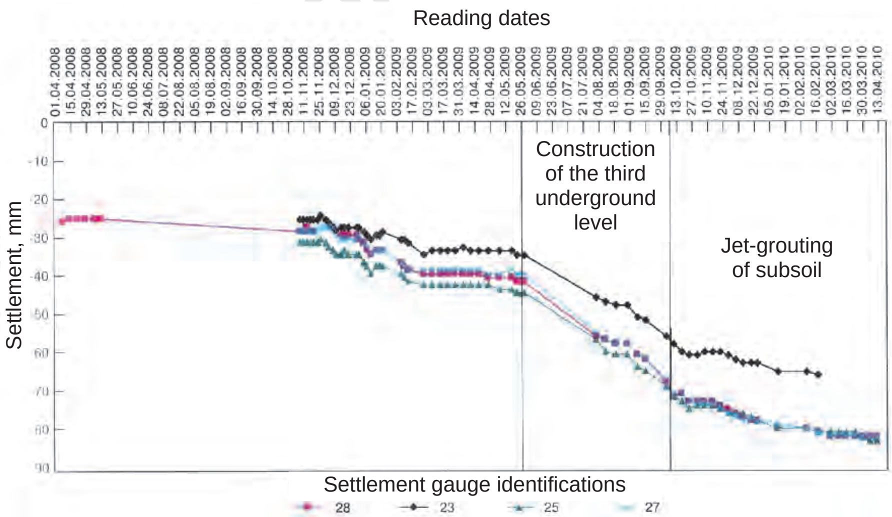


The vertical soil bulk strengthened according to the jet grouting method along the perimeter of the entire foundation pit.

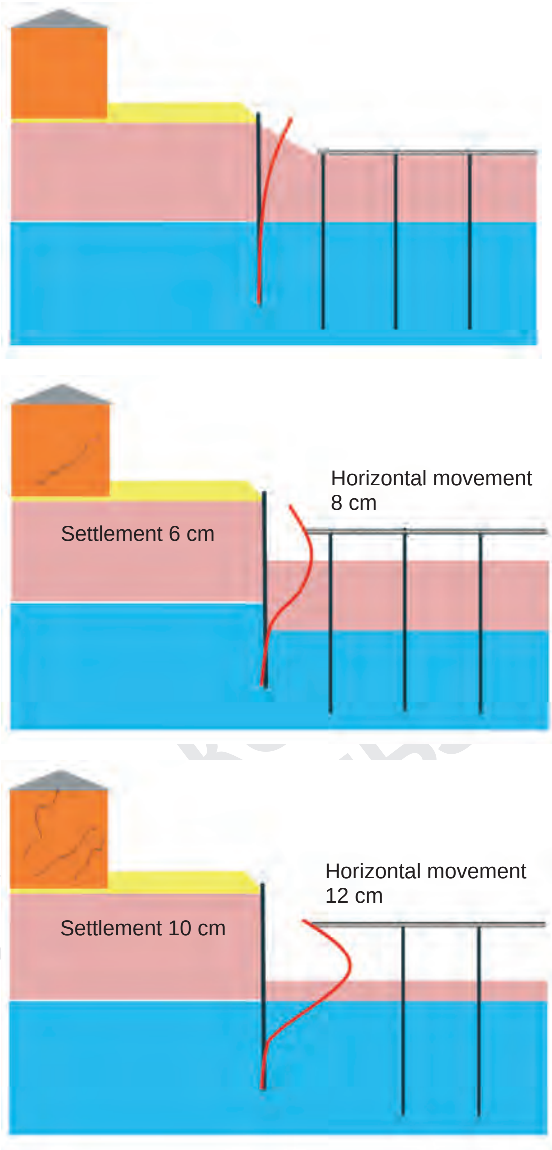


Find the 6 differences between the figures. The clues are indicated on the right: 1) only one level of shoring instead of three; 2) the layer of jet-grouted soil is not propped against anything; 3) superfluous piles are placed here and there to support the only one shoring level; 4) soil strengthened along the line of sheet piles which cannot structurally assist the sheet piles in any way; 5) mortar injected under the foundations; 6) front walls of the existing buildings underpinned with piles which transformed the houses into one-legged lame invalids.

It is interesting that the authors of the new solution nevertheless intuited the consequences of its realization. According to their calculations, additional settlements of the existing buildings triggered by realization of their option should have been in the order of 70...80 mm (please remember that the original design ensured settlements four times smaller). According to the official monitoring data, settlements of the existing buildings exceeded 80 mm all the same. According to our data the greatest settlement values exceeded 100 mm. Taking complex and expensive measures on "compensatory grouting" under their foundations also did not help; it was carried out with the intention of both stabilizing settlements of those buildings and compensating for their settlements by heaving the buildings a little, artificially. As a result the settlements exceeded the maximum permissible values 5-fold. An impressive result! Was it really *worth* changing the initial solution? Apparently, the answer is "yes", it was *worth* a lot!



The official settlement readings for the existing buildings around Mariinsky-2 site.



Retrospective analysis of horizontal displacement development of the cofferdam during excavation according to the implemented design (this figure does not show the actions which were completely useless, viz. false strengthening of the existing buildings and grouting the soil).

What was the use then of the rake walking? It was clear beyond any reasonable doubt that the new solution with the only one functional level of shoring was largely inferior in terms of rigidity to the system with the four levels. It seems that this will forever remain a mystery: over the period of design and construction two General Contractors, three Senior Designers and five Governmental Clients changed!

The whole story is like trying to identify who sewed Charlie Chaplin's suit. A bit skewed al over but impossible to find the guilty tailor because all details would have been made by a different person.

And so now St. Petersburg's architecture has been "enriched" with yet another concrete box, hardly likely to become a landmark.

Chapter 19, relating a story of geotechnical engineering without geotechnical engineers

Everyone in St. Petersburg knows a wasteland plot in the city centre located on the site of a former tram park which our valorous guards of traffic safety – the traffic police – had been using until very recently for evacuating cars parked in the wrong places. Now there is a huge construction project designed by a very well-known architect, nearing its completion. The project has a two-storied underground space under the entire area of the whole complex, in some places closely approaching the existing buildings.

Whether it was the economic recession, or overvaluation of one's abilities which is to blame, but design of the project was somehow done absolutely without any participation of geotechnical experts. Probably, because of that the project is the ideal place to invite one for another round of rake walking; the rakes exhibited there comprise some fit only for children (which, however, can strike one even more painfully at times).

The concept of underground space development, at first sight, resembles the idea realized on Mariinsky-2 project, described above in Chapter 18. However, instead of "the rigid contour" here it was supposed to arrange "a rigid island": they drove the sheet piles alongside the contour of the foundation pit, constructed the piles, made excavation in the middle of the foundation pit down to the required design depth (sheet piles being held in place by soil ramps), concreted the raft, installed struts between the raft and the sheet piles, then removed the ramps and arranged reinforced concrete structures.

The crux regarding the difference between the two projects is that a ramp is much less reliable in terms of preventing sheet piles from horizontal displacement. Many known failures in the history of underground construction in our country and abroad have been linked to overvaluation of advantages of ramps and slopes. The role of ramps in the ground conditions of St. Petersburg is especially dubious: either it is the ramps themselves that exhibit a tendency to slip, or the soft strata in their subsoils tend to lose stability. For a certain time these processes remain unseen. But in order to consider the factor of time, in order to "outrun" the threat of dangerous movement of sheet piles, one needs at least to know the rate of shear strain. One needs to know the so-called rheological properties of soil. Today it is a statement of the Federal Law on safety of buildings and structures.

Unfortunately, to comply with that requirement regarding the rheological properties it is necessary to overcome some difficulties which we mentioned in Chapter Five: *e.g.* to carry out long-term research, to develop viscoplastic model of soil behaviour and to create a computer program similar to *FEM models*.

Therefore it is not surprising that the architectural workshop did not even guess to raise the issue of possible shear strain rate modulations, which alone can support such concept of underground construction.

Obviously, having a presentiment that the real situation was more complex than it seemed, the authors of the project burdened it with such strange features as sheet piles of unprecedented size (up to 32 m), anchors of unprecedented length (42 m) with an unprecedented angle of their installation (45° – too acute), and the strengthening zone for the existing buildings of unprecedented size.

Imagine: for a foundation pit having depth of only 8 m they used sheet piles having length of 32 m! Reasonable soils on site begin at the depth of 20 m. In order to ensure there will be no significant horizontal movements of sheet piles the length of 25 m would have been quite sufficient. It is obvious that the lower 7 m of sheet piles would have never been engaged in work. In order to find this out, no complex calculations were necessary, it would have been quite enough to be able to use simple engineering methods of currently applicable construction codes. And moreover it is impossible to say that such length was used due to the safety factor. It is necessary to remember that any reliable cofferdam is arranged for the purpose of protecting the existing buildings. Vibratory hammering of sheet piles into firm clays at excessive depths is more likely to harm than help the existing buildings, provoking them into additional settlements.

Vibratory driving of record length sheet piles.

The anchors were also highly interesting. Here it is quite reasonable to remind oneself what the reason of using anchors actually is.

They are intended for protection of the existing buildings against movements of a sheet pile cofferdam. Is the attempt to "seize" soil underneath a protected building with anchors the best way forward?

In St. Petersburg, where down to depths of 20-30 m it is seldom possible to meet decent soil, the subject of anchors has never been all too popular. To reach the bearing stratum it is necessary to make anchors very long. And the longer the anchor, the greater its tensile extension. For the anchors stipulated in the design their length of 42 m at tensile extension under the 75 tons of load would yield sheet pile displacement of 4 cm. Who on earth needs such an "elastic band", not capable of preventing sheet piles from moving by 4 cm, which is already dangerous for the existing buildings!?

The methodological mistake often made during anchor tests is rather indicative also. Contemporary anchors of "Titan" type are constructed with a cylindrical coating of grout alongside their entire length. Therefore, when anchors are tested, the pull is resisted by their entire lateral surface (due to skin friction). In the working mode the anchor is intended only to hold on to unmoving soil, outside the so-called sliding wedge, which moves together with the sheet piles. It means that we are only interested in the bearing capacity of the anchor outside the sliding wedge. The question that suggests itself is: how is it possible to find it, if the test was done for the entire length of the anchor? The answer is: it is impossible to find it at all! But there is no reason to be worried about it because, in fact using anchors in that case was absolutely pointless.

Let us review, at last, the measures to strengthen the existing buildings. We, being originators of the local codes, were amazed with the actual setting of the task. The Client inquired of us: "Is it true that the TSN code says: whatever zone of influence the designer has defined, in this entire zone all houses must be underpinned? First, our designer had decided to underpin 25, and then 16 buildings." "No," – we answered, – "the designer is obliged to come up with such a solution for the new building and its underground scope that settlements of the existing buildings are within the permissible range".

To strengthen or underpin the existing buildings is necessary only in two cases: either if they are in a critical state, or if their strengthening or underpinning is obviously more expedient, than altering the entire design solution for the building under construction. The latter case is seldom realized – who will let their neighbour, who is building his new house next door, to be pocking inside their own house!? Today it is easier to suspend construction altogether. Therefore we consider it wrong to goad the investor on the way of strengthening the existing buildings. It is related to high risks of uncertainty for the investor, and capable of crossing out all possible business-plans. It would be far better for him to invest the same money in his own project in order to exclude its adverse influence on his neighbours.

Let us think together: how is it possible to strengthen an existing building if a deep foundation pit is being constructed in adjacency? At once we shall reject all speculations concerning soil improvement, compensatory injection, *etc*, suitable only for imitation of care about your fellow citizens. Let us admit we have decided to underpin the existing building with piles. And we, unlike some hapless designers, understand that it is the entire neighbouring house that should be underpinned, and not merely its part or just the adjacent wall. Otherwise the house will be transformed into a one-legged lame invalid and will crack in due course. It may be at this point that we find out that under a part of the house there are no cellars suitable for manufacturing of piles. It means it will be necessary to evacuate people from the ground floor.

Further on we shall inevitably arrive at the conclusion that having strengthened the house with piles can save it only from vertical displacement, whereas when a deep excavation is ongoing in the vicinity horizontal motions are much more dangerous, with which the underpinning piles cannot cope, just like individual rods in a broomstick. Hence, it is necessary to raise rigidity of the entire box of the building. To make this properly without evacuating residents is impossible. Thus, strengthening the existing buildings in the majority of cases inevitably degenerates into a "make-believe" measure, no one is *really* strengthening anything.

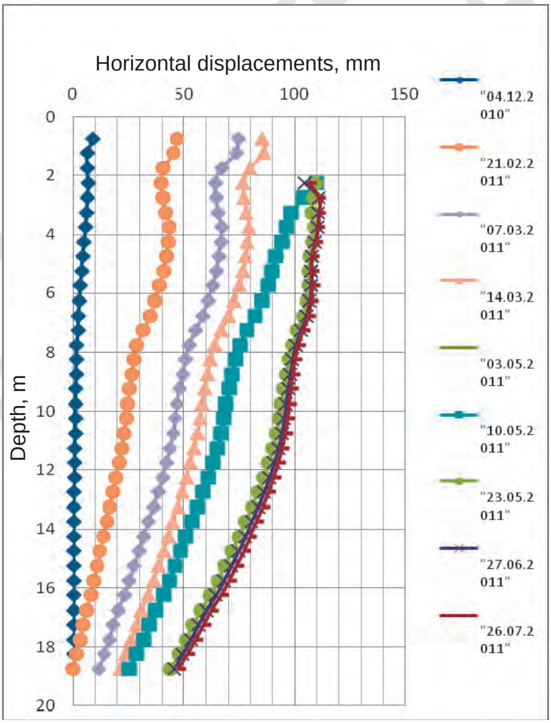
It was amusing to find that the authors of the design solved the problem of adverse influence of horizontal displacement onto the existing buildings having nipped it in the bud: in their calculations the building positioned near the foundation pit was represented in the form of little arrows of flexible load applied on the toe level of the underpinning piles. There very little remained undone, namely, to coach the building and its subsoil in the manners of good etiquette.

So, all the rakes have been carefully laid out by the designer. We shall move away, Dear Reader, to a safe distance, from where we shall be able to observe the client and the contractors sentenced to execution treading on them.

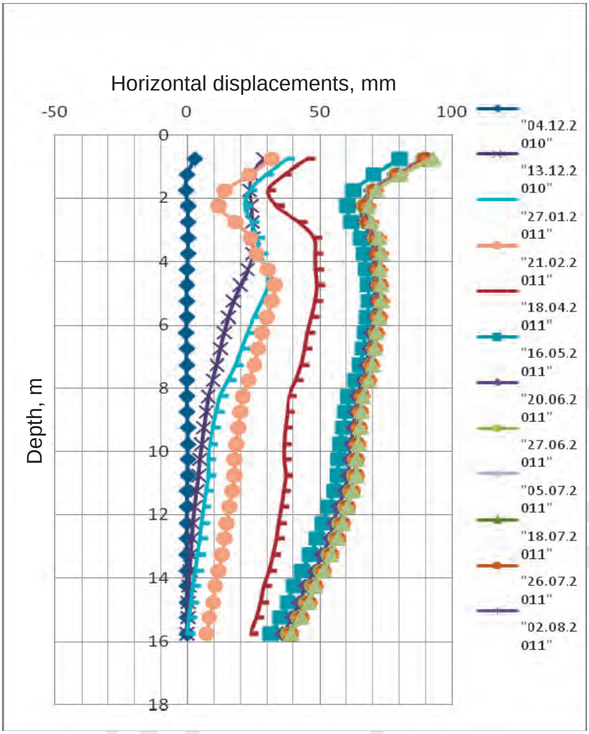
Step One. Wallop! Ouch! Vibratory driving of 32 m long sheet piles really became a selfless sacrifice on the part of the sheet piling contractor. It is just transporting such long sheet piles through the city that already merits the entry into the Guinness Book of Records, let alone their unprecedented driving! It is only a pity that the Guinness Book of Records is full largely of the most useless records in the history of humanity. In our case the trivial 25-m long sheet piles would certainly have sufficed.

Step Two. Wallop! Ouch! A foundation pit with the slopes designed to protect the cofferdam from moving into the excavation was dug. It appeared though that at absolutely different widths of ramp horizontal displacement of the cofferdam was about the same – 100 mm. Well, what was the matter? The matter was that the slope, which loses stability along the underlying deposits of soft soils, should not have been used as the prop to protect the cofferdam.

Excavation with slopes.



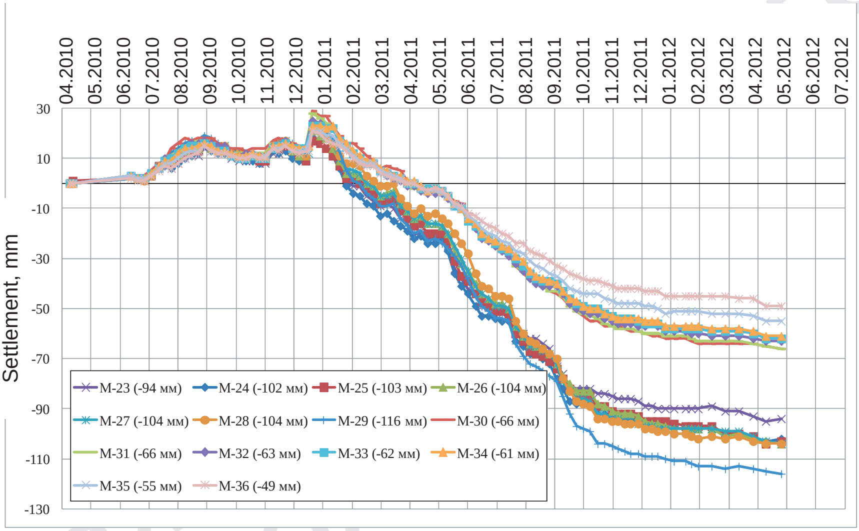
Horizontal displacement curves for the section where the cofferdam was protected with ramps.



Horizontal displacement curves for the section where the cofferdam was protected with both ramps, and long anchors.

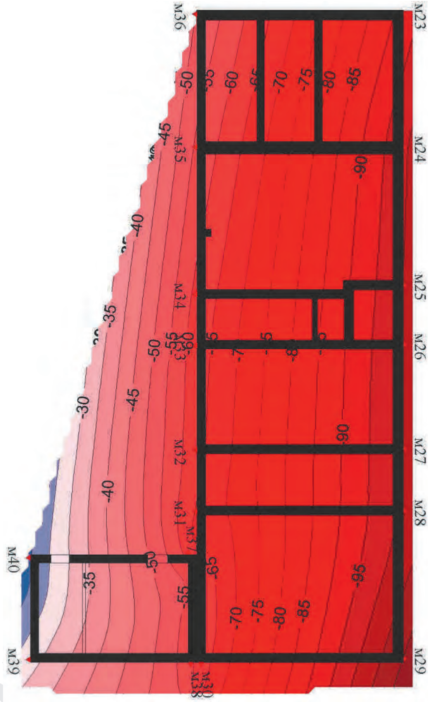
Step Three. (Well, enough of those sadistic sound imitations…) When the pit had been excavated it was found out that the cofferdam with anchors had moved inside the pit by the same 100 mm!

Step Four. Before we observe, we should point out that fortunately, the excavation closely approached only one building, the others being at distance of 15 m and more. Therefore the designer's entire power of attention was concentrated on rescuing that house. First, along the building's border a wall of secant piles of small diameter was constructed. This was followed with *Arcelor* extended type sheet piles. And then behind the sheet piles a thick solid diaphragm wall was placed. While all this was being perpetrated and the ramp-protected bulk excavation was being dug, the rescued house settled by the same 100 mm. There is nothing mystical about the sameness of the value. Each technological operation performed in the ground near a building leads to settlements of the latter. Besides it is impossible to ensure immovability of a cofferdam by means of simply increasing its thickness. Reliable horizontal struts (instead of a creeping ramp) are essential.

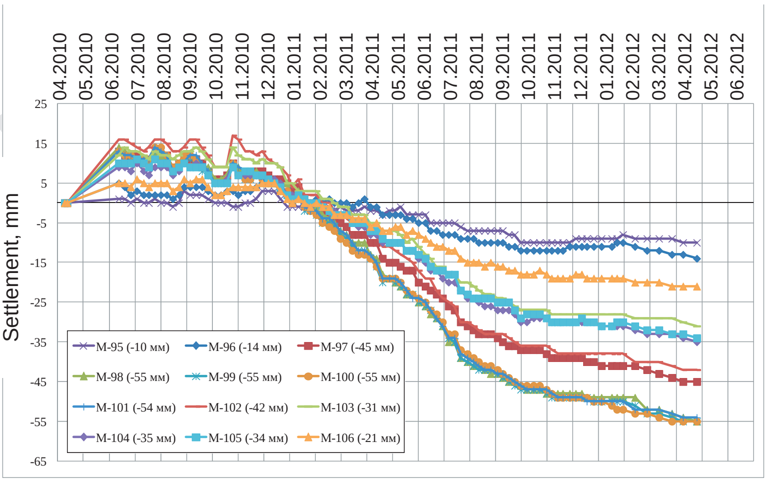


Development of settlement in time – the building directly adjoining the foundation pit: first the house had been lifted by 30 mm due to construction of multiple bored piles in its adjacency, then while it was being rescued it sank by 110 mm.

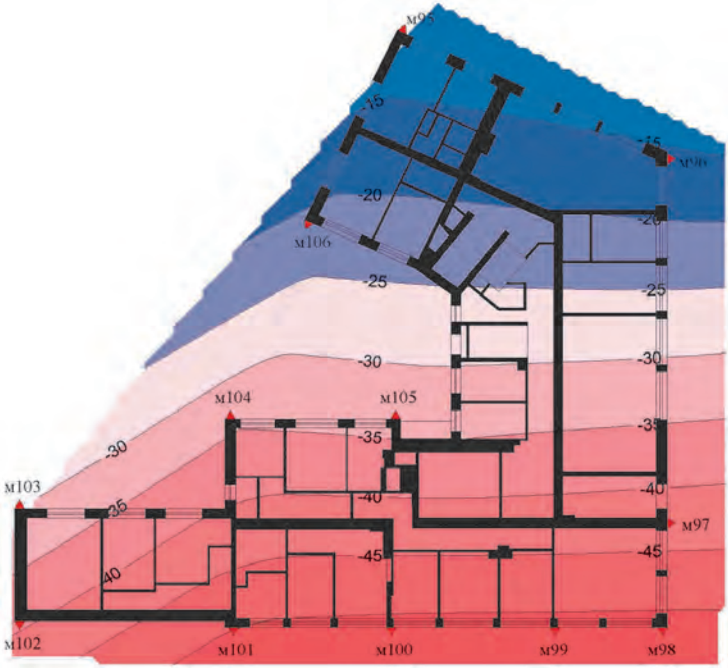
The settlement trough thus formed pulled inside even the houses located at distances of 25 m from the foundation pit – their settlements exceeded 50 mm!



Settlement contours of the building directly adjoining the foundation pit.



Development of settlement in time – a residential building located at 25 m from the foundation pit: first the house had been lifted by 15 mm due to construction of multiple bored piles in its adjacency, then it settled by 110 mm.



Settlement contours of a residential building located at 25 m from the foundation pit.

It is doubtful that this rake walk gave anyone any pleasure. Whether the designer actually learned any lessons from it, we cannot tell. But we precisely know that there can be no geotechnical engineering without geotechnical engineers, as there can be no architecture without architects.

It seems it was the Russian fable-writer Ivan Krylov who said: *"There would be trouble if the cobbler made the foods, and if the cook, instead of cooking, mended boots".* One should never really forget the Russian classics, even those not related to geotechnical engineering.

There is an old paradox: "the bigger the circle of knowledge the longer the border with the unknown". Those whose knowledge (say, in the field of geotechnical engineering) is limited, may not know how much they actually do not know.

Chapter 20, relating the story of a foundation pit on the Chinese-Finnish border

Alas, but the old Russian joke about everything being perfectly serene on the Chinese-Finnish border reverberated in our city, producing an absolutely serious building with an underground scope in the city centre. The Finns invested the money, and the Chinese spent it on construction. And all would have been perfectly serene and would have worked out absolutely without "the natives", had it not been for St. Petersburg specificity – the complex ground conditions. Here you cannot do without local experience (thus speaketh the Eurocode even!)

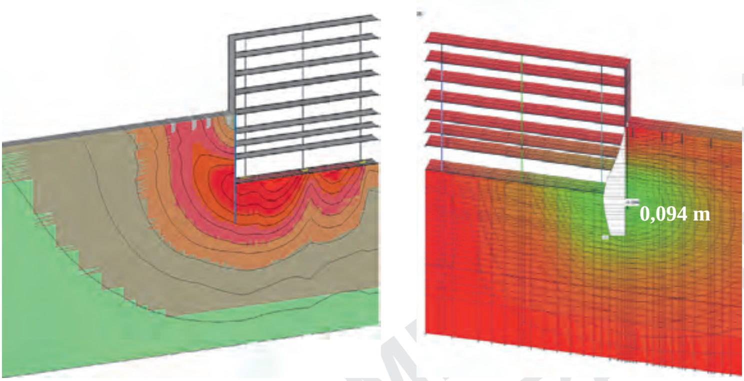
The challenge was extremely complicated: it was necessary to construct a four-storied underground parking with depth down to 14 m and the plan dimensions of 138 by 87 m immediately adjacent to existing historic buildings. For the existing buildings not to tumble down into the foundation pit, the cofferdam should have had no deflections or displacements. Each centimetre of cofferdam movement would have led to a gain of settlements of the adjacent house equally by one centimetre. Hence, the cofferdam had to be: *a* – rigid, *b* – strutted, *c* – safe in manufacturing.

Already at the preliminary stage of design examination it became clear to us that for these purposes the proposed sheet pile cofferdam, even if shored in several levels according to *top-down*[1] technology, would be no good. First, sheet piling is flexible. Sheet piles of the most peculiar shape hardly reach rigidity of a reinforced concrete diaphragm wall with thickness of 800 mm. Secondly, to drive sheet piles to the necessary depth it is necessary to apply high-frequency vibration which is hazardous to health of the existing buildings and its inhabitants. Here a diaphragm wall, perhaps even with buttresses

[1] *Top-down* is a construction technology when the building is constructed simultaneously upwards and downwards, using the floors of the underground section as cofferdam supports (see previous chapters).

considerably reducing its possible yield, was required. A diaphragm wall plus shoring according to the *top-down* technology – this was seen as a guarantee of safety of the existing buildings.

It is necessary to mention the fact that by commencement of construction the technology of the monolithic diaphragm wall had already passed the stage of approbation.



Calculations suggest ground displacement along circular-cylindrical surface; sheet piles move by 9.4 cm, the existing building develops settlement of the same order of magnitude.

Unfortunately, these simple reasons were not heard by the developer, he did not want to know. He had already purchased the sheet piles, and, consequently, for him safety issues for the existing buildings somehow faded into the background by themselves. The designers tried to compensate for insufficient rigidity of the cofferdam by construction of one more shoring level of jet grouted[2] soil below the bottom of the foundation and underpinning the existing buildings with inclined piles. However with flights of 80 m, strutting in the form of a 2-meter layer of fixed soil works badly and if it is

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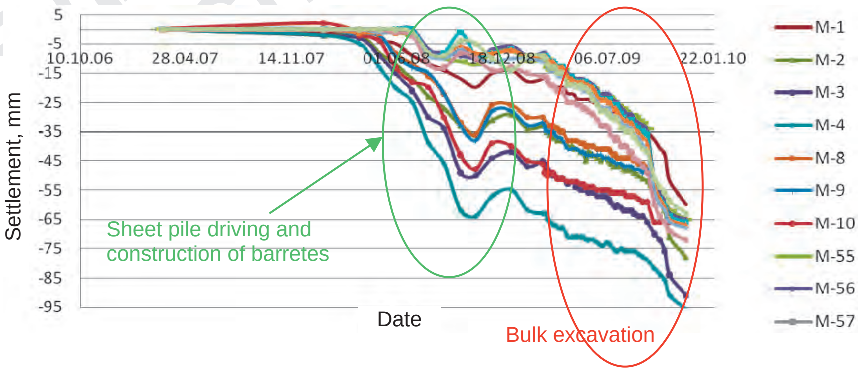
[2] *Jet-grouting* is a high pressure jet injection technology, using a jet of cement mortar leaving the nozzle of a rotating monitor lowered into a borehole; injection is made at pressure levels of 300 atmospheres. It mashes soil into a pulp which it forces overground replacing with cement mortar. The resulting product is a cylinder of cement with soil additives. It is 100 times stronger than soil, but 100 times weaker than concrete (see previous chapters).

made fragmentary and patchy it does not work at all (it was exactly that option that had been adopted by the economical client). Strengthening of the existing building was constructed of inclined underpinning piles (working also as bending elements) with centrally placed reinforcement cage. Neither domestic nor foreign codes allow such solutions, because the capacity of such pile's material to withstand bending is negligible. It is amazing that so obvious an infringement of elementary requirements of design codes, which proscribe designing piles as if they were reinforced concrete structures, is so frequently encountered in underpinning works practice.

"It is at least necessary to try out the design solution on a test pit", – the State Expert Board coyly suggested. "What for? – retorted the developer. – If the test pit fails what shall I do with the sheet piles? We shall experiment on the existing buildings!"

And here is the expensive result of the experiment. The adjacent house underpinned with piles took a 95 mm settlement plunge. Another, whose owners opposed the underpinning, settled by 70 mm. It is a very clear illustration of the "advantages" of inclined piles with the centrally positioned reinforcement.



Settlement history for the adjacent house strengthened with underpinning piles.

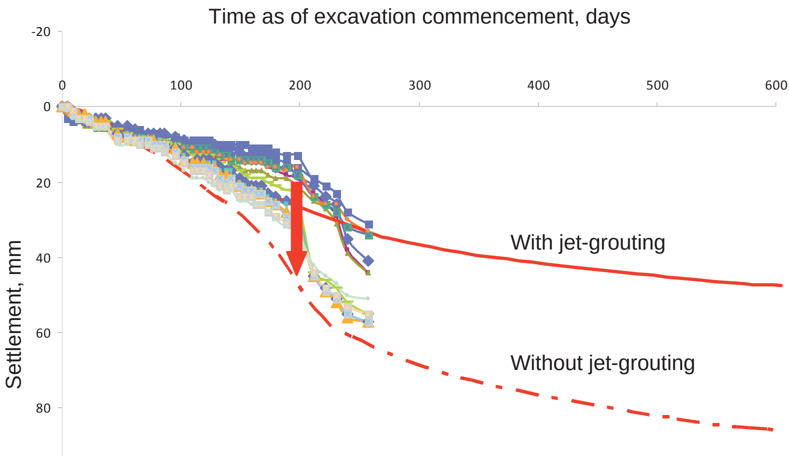
It is necessary to say that the "achieved" result, 5 times exceeding the settlement limits permissible in our codes, is a VERY happy end for this project. And here is why.

When insufficiently rigid cofferdam has been used there is a unique chance to keep the existing buildings intact and this chance is extremely fast excavation of the foundation pit. The faster they excavate one floor and arrange the shoring disk, the less the cofferdam will move into the pit and the less the existing buildings will suffer. For a conscious choice of such a dangerous principle of works, it is necessary, at least, to know the rheological behaviour of the soil on the site (in more simple terms the speed of its deformation). It is necessary to calculate by how much the cofferdam will move under soil pressure, over the time required to excavate each underground floor when the hardening reinforced concrete of the shoring disk engages in action. It is our deep conviction that success of construction should not be based exclusively on nimbleness of the contractor, especially, when it is not known, what the rate of strain of soil is.

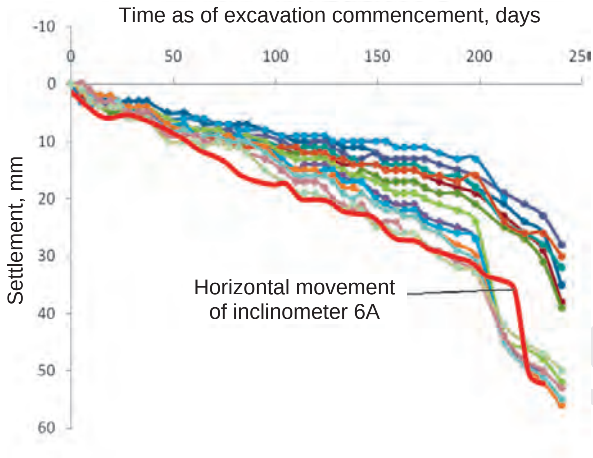
It was in this difficult situation that we found the poor contractor. His international advisers echoed us: it is dangerous to realize a project in which there are no tools to withstand development of the emergency scenario. It is necessary to say that the contractor understood what he had got himself into, but there simply was no way back. We thought for very long how to help the contractor to construct a house with an underground parking, as the client wanted, and at the same time not to endanger the residents of the existing buildings. And we found a solution!

The missing data on the rate of strain was possible to receive during construction of two higher underground floors when the risk for existing buildings is still insignificant. For this purpose it was necessary to measure horizontal displacement of sheet piles and settlements of the existing buildings. After that, using back analysis, it was possible to define the missing rheological parameters of soil. And that was the way we did it. The calculated parameters, which define the rate of strain development, were further used to predict what can happen after, *i.e.* during excavation for the two remaining underground floors. As a result the contractor received the unequivocal advice: he had to excavate with terrible speed if he did not fancy digging the (no longer) existing buildings out of his foundation pit.

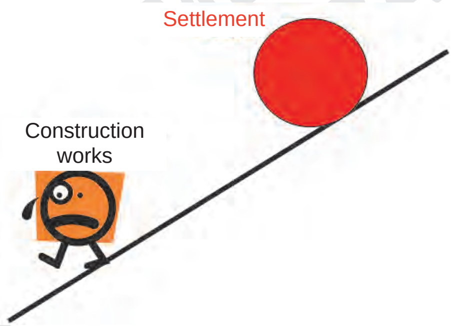
Here you will not envy the contractor, who, like a mountain skier, is flying down the slope chased by an avalanche. If you get distracted for even a moment the wave of irreversible movements in subsoil engulfs the fruit of your works. *The fountains of the great deep*... It has to be given to the brave contractor: the existing buildings were saved – they settled "only" by 95 mm and remained usable. But this case cannot become an example for emulation in any way.



Comparison of our settlement prediction for the existing buildings and the real observations. The arrow indicates the moment when the "anchor" of strengthened soil began to move together with the cofferdam.



The horizontal deflection of the cofferdam (red line) provokes settlements of the adjacent house (a bunch of multi-coloured lines). Approaching the foundation pit the value of settlements comes nearer to the value of horizontal displacement of the cofferdam.



On the contrary, it is an example for the State Expert Board too, who are not particularly partial to actually checking calculations, and for the Construction Supervision Committee who is sometimes over-inclined to protect the developer from importunate residents of the existing buildings. These organizations, created to protect the city from ruin, at this given project behaved rather modestly and timidly. Probably, this was because the Construction Supervision Committee, in the person of its affiliated structure decided to assist the developer and was the monitoring contractor for the project. The affiliated structure established that all was well, and that the existing buildings themselves were to blame for having lived to an old age, which is why they cracked (and nearly croaked). And the settlements were not at all 95 mm, but only 50 mm, they needed to be accounted for not from the very commencement of construction, but from its middle, in order not to spoil the beautiful figures in the reports. Because who will blame the Construction Supervision Committee that they did not timely interfere with the situation? God forbid that we should see here the notorious conflict of interests between a punishing hand (Construction Supervision) and a hand earning its bread in the exhausting competitive struggle in the field of site monitoring business (Affiliated Structure). For it is highly difficult in the father-like manner to rebuke those who pay the piper…

Chapter 21,
relating the story of the sunk shaft, which really sank...

When building underground engineering structures, services, *etc,* in greenfield conditions the classical technology of the so-called “sunk shaft” is often used. This technology is as old as the world: you just take a spade, get your person inside a concrete ring, do some digging – and the ring goes down by itself as you dig. In the same way it is possible to sink a heavy large diameter reinforced concrete ring, you just use machinery for digging. This way has been used to arrange lots of underground structures of various diameter and even rectangular in plan. There have been also huge shafts constructed, more than 100 m in diameter. Experts of “Georeconstruction” Institute have designed more than a hundred of similar structures, sometimes in very complex conditions, for instance in the middle of a river on a sandy island.

But in cases like this, what use is there really for experts, it might be much more interesting and, let’s not deny it, less expensive without them, mightn’t it? That is why the story of the sunk shaft which we shall relate to you is most fascinating. The size of the shaft was rather modest: diameter – 20 m, depth – 12 m. It seemed its construction should have caused no problems: enormous experience of building such structures had been accumulated. But, no...

Sunk shaft of a sewer pump station (appearance following the underpinning with bored piles and the onset of tilt up to 1200 mm).

From the very beginning, things started going pear-shaped. Remember, in Chapter 3 we said how important it is to correctly define the soil conditions. The troubles of the ill-fated shaft began with the geology.

The client, apparently, should have known better about trying to scrimp on site investigation – the surest way to spend an awful lot more money correcting building mistakes, a kind of false economy. However, in practice the desire to save on geology is an ordinary phenomenon, because, you know, economy belongs to "here and now", and possible problems are a thing of an uncertain future. How is it possible to do site investigation more cheaply? The most simple and popular way is to do without soil tests. This is done in a very simple way: the density and humidity of soil are measured, whereas mechanical properties (necessary for geotechnical calculations) are invented. Unfortunately, this way became so trite that some grifters who decided to work in the field of geology had a desire to go further. However, further "economy" acquires unequivocally criminal character. Here are some cons the grifters use: 1 – make boreholes of smaller length and continue them on paper from your imagination; 2 – do not spend energy on drilling boreholes, instead take an archive borehole somewhere in the neighbourhood and draw something similar. It is like treating a patient using medical specimens of a man who lives next door. We call this practice "pencil drilling". One firm – we shall call it "Firm B" – has reached soaring heights in the business of "pencil drilling". We had spotted their site investigation forgeries some time previously and registered them in our personal "black list".

First time we encountered those conmen about 10 years ago on a project near underground station "Pionerskaya". The hammer-driven piles on the site could not be driven to the design toe level, it appeared – for some mysterious reasons. Looking at the geological profile prepared by "Firm B", it was absolutely impossible to understand what the matter was. Having drilled a test borehole, our geologists found a rather conspicuous stratum of shingle and eight soil strata layers instead of the indicated four. Well, there can of course be some mismatch about how you call the geological strata that you have found, but not to notice a layer of shingle which ruins your drilling bits when you hit it is quite another matter and was simply impossible.

Another time "Firm B" distinguished itself on Moskovsky Avenue having indicated that they had drilled a deep borehole when in fact it was much shorter. This was found out when a test hole was drilled in the same place during further investigation. According to "Firm B", there was moraine at the depth of 30 m but actually Cambrian clay began at about 25 m already.

Well, that kind of "invented geology" of "Firm B" resulted in the most fatal consequences for the ill-fated sunk shaft project. For they had drawn a moraine stratum at the depth of about 10 m where the sunk shaft, admittedly, should have stopped sinking.

Generally speaking, "Firm B" should have known at least the general features of our geological conditions. There was a river near the sunk shaft site. And it happens very often that rivers prefer to choose locations of paleovalleys. The Neva, too, flows above paleovalleys, dug through Proterozoic deposits by prehistoric streams. The paleovalleys over millions of years got filled with highly variable and, as a rule, rather soft soils. Therefore "inventing" geology near a river one really has to have remarkable bravery, which might be better used elsewhere.

When during sinking of the shaft it was realized that having reached the depth of 12 m it was not stopping, it was necessary for "Firm B" to arrive to the site and to finally drill the boreholes properly. It was discovered that the moraine began at depth of not 12 but more than 30 m! There had been a "slight" error, you see...

Especially amusing is the outlook of soil stratification prepared by "Firm B": it appeared that the shaft with a sharpshooter's accuracy had landed directly into a "crater" with slopes of amazing steepness - 70^0. Such steepness does not exist in St. Petersburg region, it is absolutely improbable. The reason for the "appearance" of the "crater" is simple: "Firm B" was not brave enough to admit their adherence to sharp practice.

The "crater" in which the sunk shaft sank, as a result of "pencil drilling".

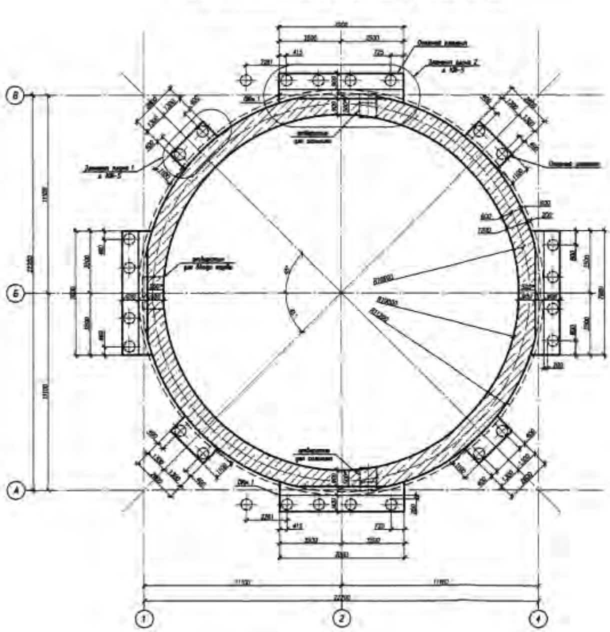
The designer, who was not adept at geotechnical engineering, did not suspect the dirty trick until the fictitious geology had led to sad consequences. They had not had enough time yet even to excavate down to the necessary depth, as the reinforced concrete ring continued to plunge vigorously down. And it was perfectly clear that according to the corrected geological profile it would never stop by itself.

The question was asked immediately: *what can be done!?* Again, the answer was obvious: we could analyse the situation and modify the design, and for this purpose experts had to be involved. However, here another typical mistake was made: to correct the problem they approached a contractor. We already spoke in the beginning of the book that asking contractors to do design is impossible, because the contractor will "prescribe" not what is necessary, but what he has at his disposal. In the given situation they stumbled upon a respectable company who specialized in bored piles...

As they say, "this is a small world", but the professional world is even smaller. A former graduate of ours told us the entire story. As soon as we looked at the geological profile it became clear that piles would be useless. Above the moraine deposits down to about 30 m there were soft soils of potentially liquid consistency. Imagine you immersed a bottomless barrel into a liquid. It is clear that if you place the barrel on piles it will impart to it some stability, but will never help to pump the liquid out of that truly "bottomless" barrel. If the liquid in which the shaft is immersed is viscous enough, then as it flows inside the shaft, the piles will follow downwards and inside, and eventually, break.

However, when a firm's backyard is full of idly standing piling rigs, the question of their efficiency in the given situation becomes somehow inappropriate. It was possibly on that account that the decision was taken to strengthen the shaft with piles. They constructed both piles and pile caps – massive "handles" attached to the concrete "casserole" without a bottom.

Arrangement of the shaft's pile caps



A fragment of design for underpinning the shaft with bored piles.

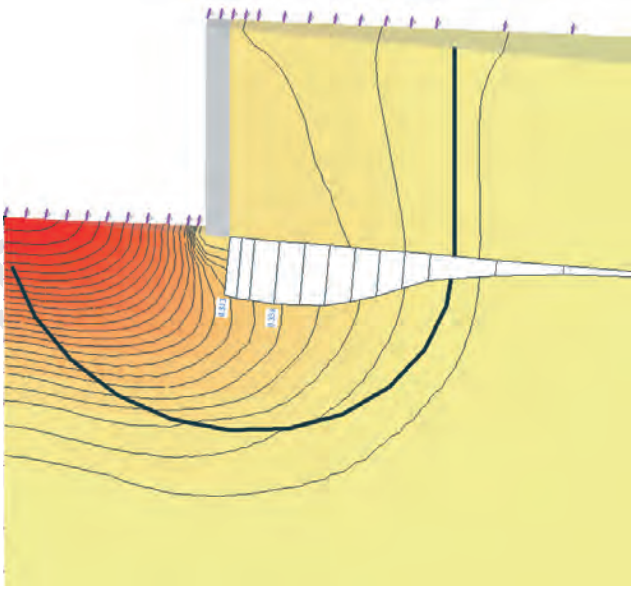
In the fluid remoulded soil most piles ceased to work and due to negative friction sank into the subsoil, advancing downwards ahead of the supporting elements of the shaft itself; in several places there were gaps between the supporting elements and the pile heads.

At this stage one more mistake came to light: this time the surveyors were to blame – they got confused about the levels of the shaft, having made an error of about 1 m. It needs to be said that all the mistakes in this story were made on a grand scale, no one wasted time on being just a little wrong. As a result of the surveyors' mistake it was thought that the shaft had not yet reached its design sinking mark, whereas before it was believed that it had already gone too far down. The construction superintendent decided to sacrifice the sections of the piles to be embedded into the pilecaps and to cut the piles on the necessary level. As the excavation continued, the shaft sank along with it and safely landed with the pilecaps on the pile heads.

But after, it all started happening according to our scenario: at the last stage of excavation the soil, naturally, continued to come inside through the missing bottom. Problems appeared with the concrete blinding. Then, one night, the shaft tilted, at once almost by 1 meter. Soon the tilt reached 1.2 m, and the shaft got filled up with soil by almost one third of its volume. We found that unfortunate shaft already in the warped position, as shown on the first photo in this chapter.

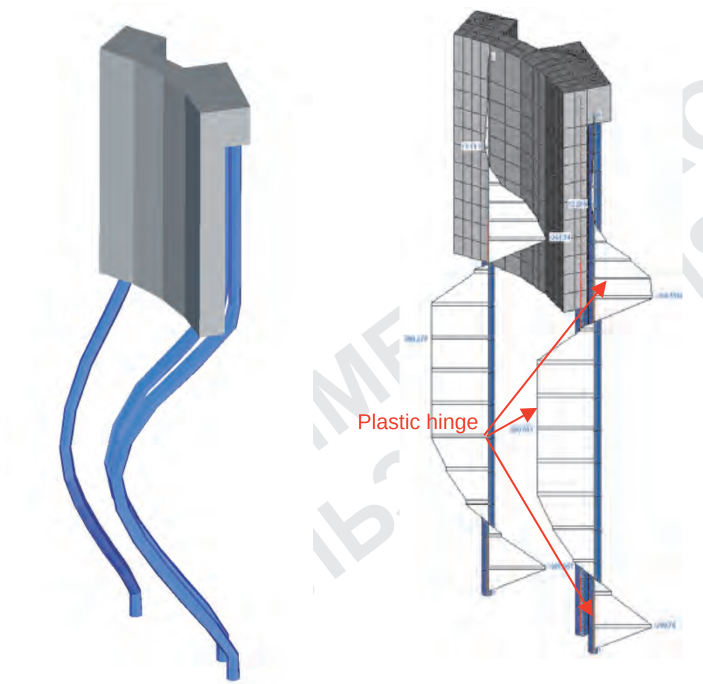
First of all, it was necessary to analyse what had happened. Initially we performed research of soil properties ourselves, as we could not trust the former geologists, but then again, they had never done the laboratory tests, necessary for calculations. We computed soil behaviour during excavation by means of two programs, and also invented a special analytical method of calculating the given situation. All methods of calculation showed that to excavate the soil inside the shaft to the design level was impossible. There was a constant loss of stability in the soil, it kept flowing in. Indeed, inside the shaft the ground was bulging upwards, whereas around the shaft subsidence was clearly visible. We shall notice here that analysis of such emergencies is very useful to estimate reliability of various calculative and computing methods in comparison with the really observed situations.

The piles actually behaved in a very odd way. When a pilecap was excavated, it was found out that one of the two associated piles had "escaped" downwards, moving ahead of the settling shaft body. It appeared that that situation too quite conformed to the calculation results. Modelling the problem with piles it was established that they in no way could keep the ground from flowing inside the shaft. It is only an illusion that reinforced concrete 620 mm diameter piles are a rigid structure. Actually "the inner voice" here works badly, as we are not capable of imagining the magnitude of forces which are transferrable to the piles in this case. Calculation demonstrably showed that the ground was pulling the piles along with itself inside the shaft, bending and, eventually, inevitably breaking them, like matches. During the "pulling" of piles inside the shaft one pile simply "outran" another in a pilecap.



The scenario of ground stability loss around the shaft. Numerical solution by means of FEM models. The safety factor is less than 1.

Obviously, the piles got broken not all at once, too. First, the piles on one side of the shaft had fallen victims to the situation, and it was in that direction that the unfortunate structure tilted. The piles which had had no time to break had a manifold loading increase. As a result, the pile started to break out some fragments from the wall sections of the shaft (imagine the power required to make a crack in a reinforced concrete wall 1.2 m thick).



The scenario of stability loss in spatial setting and the mechanism of pile destruction.

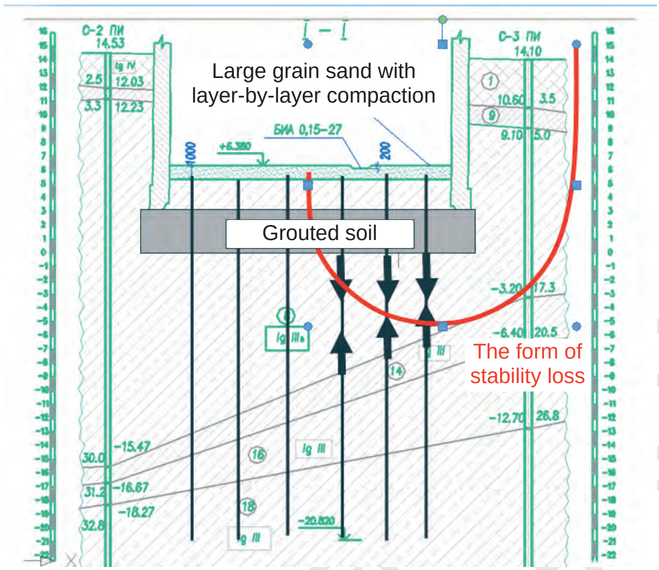
So, the calculations showed that underpinning of the shaft with bored piles would not have worked anyway. Such analysis, obviously, should have been done prior to the underpinning actions – then it would have been possible to prevent the tilt. However, resulting from "the well co-ordinated actions" of the entire Mickey Mouse team, of which the geologists were the first, and the underpinning designers were the last, the sunk shaft turned out, firstly, tilted (by 1.2 m), and, secondly, cracked.

The task to save the structure, set before us, was not of the easy kind. First of all, to prevent further sinking the shaft had to be backfilled. According to our calculations it was required to add about 3 m of soil inside the shaft. When the backfilling material had been delivered on site, you should have seen the face of the construction superintendent, who had already tried twice to excavate it…

Secondly, it was necessary to "equip" the shaft with a kind of bottom so that it could finally be dug out to the design level. And to accomplish that was possible only in one way – using jet-grouting technology. But the jet-grouting technology alone was not enough. The layer of soil-cement mixture, even if 3 m thick, was a bit on the thin side for the concrete shaft of 20 m diameter. It was impossible to reinforce the soil-cement layer with horizontal rebar, which meant that during excavation it would inevitably be broken by the pressure of soil pushing up from below. In order to rule this out we anchored the soil-cement layer in the dependable low-lying strata (you might say "nailed it down" to the moraine). Again, the developed analytical and numerical methods really helped (they had been practically tested in the given situation as they allowed to simulate all the previous stages of its development). We calculated the load which should have been assumed by the anchors for the bottom not to break and not to start climbing inside the shaft. As a result our design made it possible to excavate the shaft and to construct the bottom slab, this time of reinforced concrete.

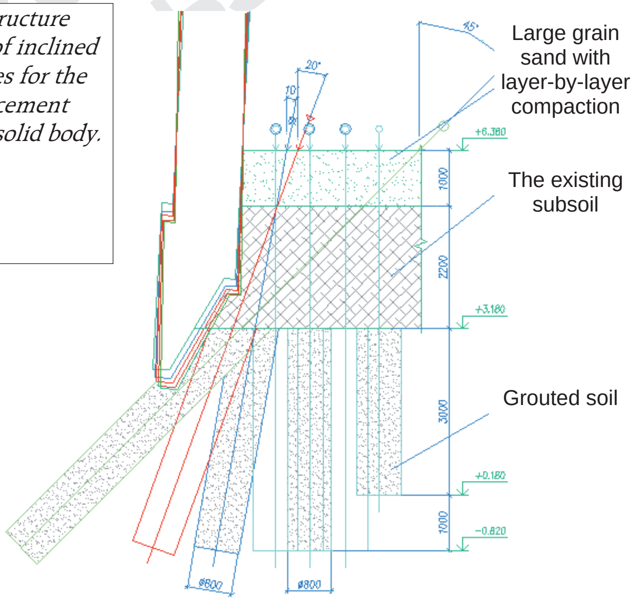
The moral of this story is simple and obvious. First, do not save money on site investigation. Firms like the above-mentioned "Firm B" should not even exist on the market at all. Today, on the contrary, they win in the competitive struggle because they offer lower prices. Indeed, "pencil drilling" is cheap and quick.

Second, to rectify a critical situation do not hire a contractor (whose *raison d'être* is to sell you whatever he has to sell), but hire an independent expert-designer (who will tell you what really needs to be done).



The anchoring arrangement for the prevention of stability loss of soil and its intake inside the shaft.

Underpinning the structure required a network of inclined and vertical boreholes for the arrangement of soil-cement columns, forming a solid body.



Third, the geotechnical science yet again proved its efficiency. What had to happen according to the calculations, happened. It was, certainly, a positive result for us, geotechnical engineers. But for the client it was not. It appears that the initial cost of the shaft was exceeded several times. This proportion of possible loss should be remembered for one not to be tempted by infinitesimal savings on site investigation and the design.

Part Six

Thematic guided walks through remarkable sites

Chapter 22. Walk One. For crime drama enthusiasts. Investigating causes for deformations of buildings.

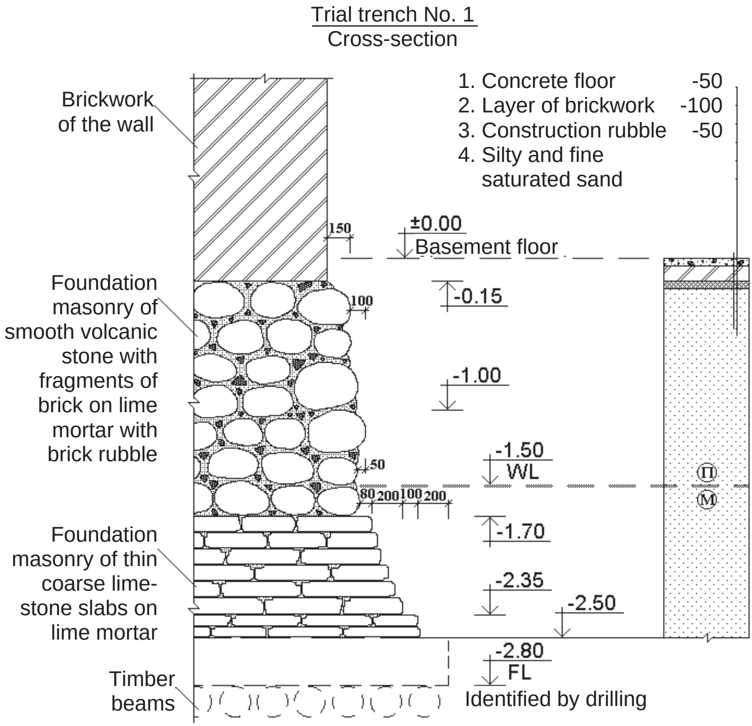
Deduction methods, popularized by detective novels, can prove highly practicable when causes for deformations of buildings need to be established. Without knowledge of those causes it is impossible to give the right diagnosis (if at this point we may be permitted a medical idiom again) and prescribe appropriate treatment. Looking for the causes themselves, however, is more akin to a job of a detective inspector.

St. Petersburg Stock Exchange on the Basil Island Spit

In 2002 the entire city of St. Petersburg was preparing to celebrate its tercentenary. Streets, squares and elevations of buildings were being given a grand facelift. Among them was one of the major city landmarks – The Stock Exchange on Vasilievsky (Basil) Island Spit. The building was constructed in 1805 as designed by *Jean-François Thomas de Thomon* in place of an earlier edifice by *Giacomo Quarenghi*. The front elevations of the Stock Exchange Building were given special attention to mark the tercentenary. But much to the restorers' chagrin, new cracks kept appearing on the newly plastered and painted walls. Especially galling was a stubborn crack alongside the building's axis above the semicircular window on the pediment. How dangerous were the cracks? How would they affect the building's future? Was any strengthening required on the front hall vaults or the foundations? The Russian Federal Agency for Construction (Gosstroy) commissioned "Georeconstruction" Institute to answer those questions. A competent inquiry was launched looking into all possible "scenarios". Primarily, it had to be investigated if a certain value of settlement differential might have been programmed even as early as the construction stage, *e.g.* by using the pre-existent Quarenghi foundations to erect the new superstructure. Such had been often the case in construction history – suffice it to mention St. Isaac's Cathedral, where various types of foundations had been used, partially retained even from the original Rinaldesque structure. This possibility was thoroughly explored through both a study of the documents and a comprehensive *in situ* investigation. We reproduced the sequence of foundations construction. It turned out that, at first, a strip foundation for the outer stylobate walls had been constructed, consisting of granite boulders and limestone slabs and forming a rectangular dam.

The Stock Exchange Building at the moment of construction completion.

The dam then served to protect bulk excavation, in which a bedding of timber beams was laid, followed by a solid limestone slab topped with pillared rubblework foundations. Apparently, the "non-homogenious foundation hypothesis" was not true. The timber beams had not rotted, the silty sand strata underneath being of dense consistency. Correspondingly, the observed deformations could not have been related to such typical St. Petersburg phenomena as decomposition of timber elements or suffusive decompression of underlying sands.

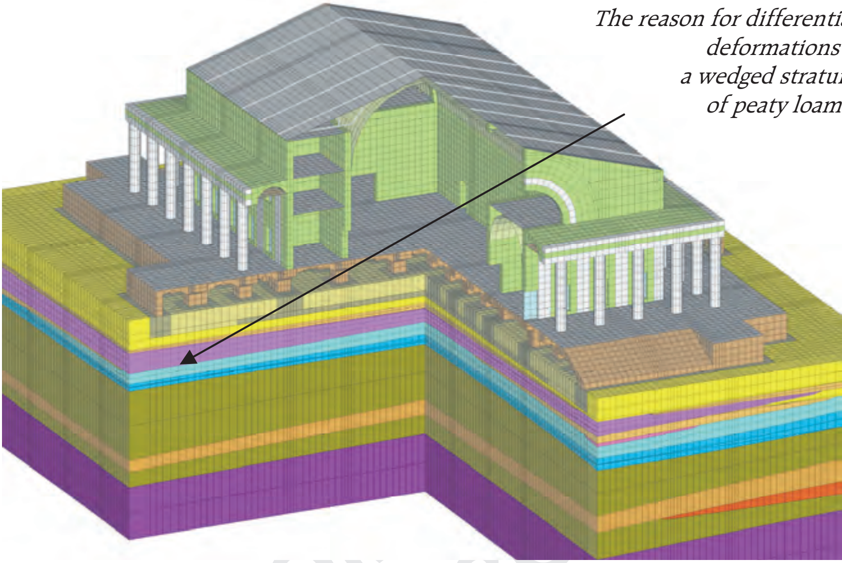


The foundations of the external wall of the Stock Exchange stylobate.

Other version stated in a number of earlier inspections was the assumption that the deformations were connected with unbalanced equilibrium in the central vault. Inspection of structures rejected this hypothesis also, as being unreasonable. It appeared that during reconstruction of the Stock Exchange in 1914 according to the design of F. Lidval a false vault was arranged, constructed of consoles suspended from reinforced concrete trusses.

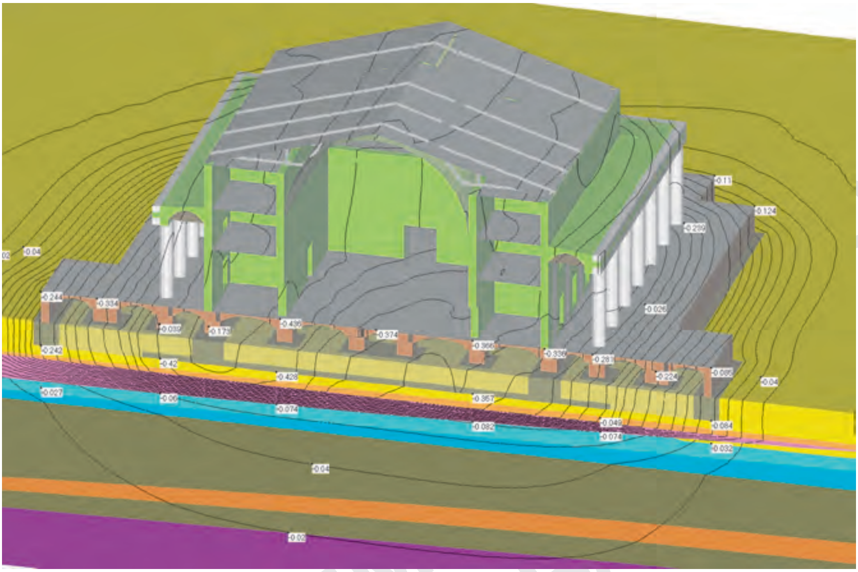
As a result the list of hypotheses was reduced to only one possibility: heterogeneity of ground conditions on site. Our geophysical investigation (seismic tomography) confirmed presence of weaker soil under the southern part of the building.

How was it possible to check this hypothesis? Here our *FEM models* software came to help us allowing to model interaction of the building's superstructure and its subsoil.



A fragment of calculation scheme for the Stock Exchange and its subsoil.

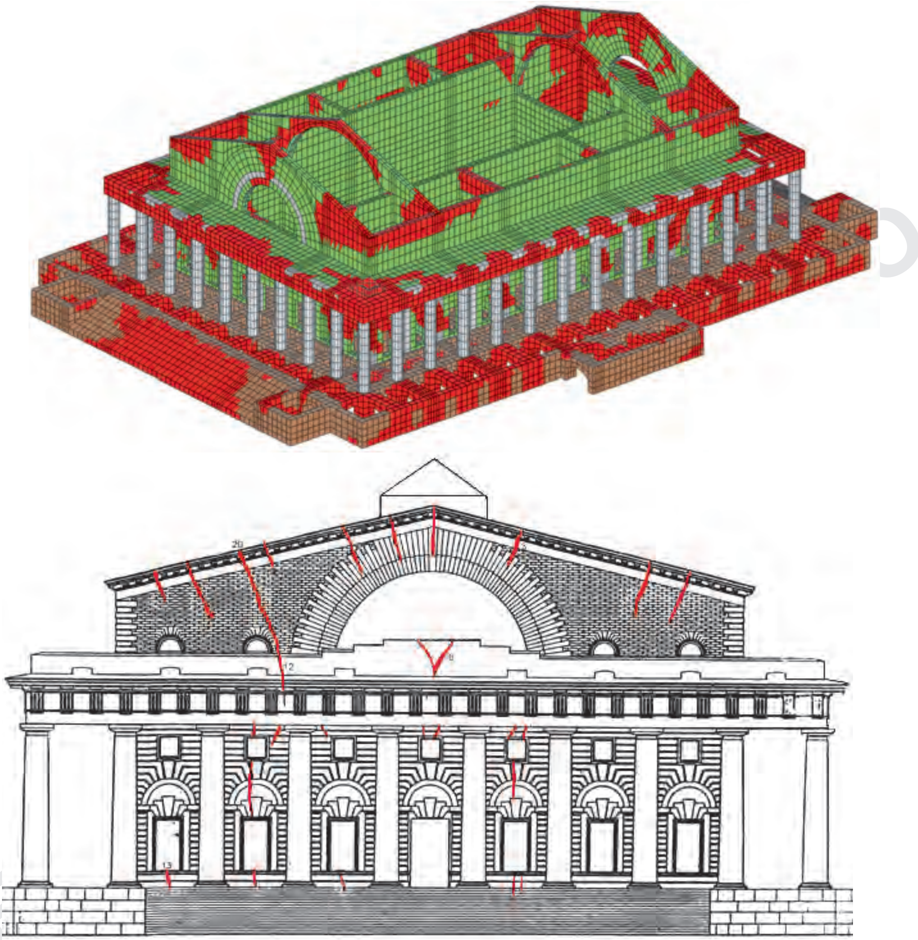
As the calculations showed the settlement of the building occurred unevenly: over the period of its existence the southern façade settled by 43 cm, and the northern by 28 cm. Settlement differential according to calculations (15 cm) quite corresponded to geodetic measurements. Certainly, no initial geodetic supervision over the Stock Exchange from the moment of its construction was conducted, but an idea of the accrued differential could be obtained having carried out surveying of the bases of the columns and the pilasters on the entire perimeter of the building. As a result of measurements it was established that the southern façade had settled 13...14 cm in excess of the northern.



Settlement contours of the Stock Exchange according to soil-structure calculations.

The greatest contribution to development of differential settlement was brought about by the stratum of peaty sandy loams. It sits as a wedge under the Stock Exchange building: under the southern façade its thickness is bigger (up to 3 m), whereas under the northern facade it is only 0,5 m.

Because of development of differential settlement, within the building's walls zones of tensile stress appear. And where there is stretching in the brickwork, there it cracks. The settlement profile agrees quite well with the observed layout of the cracks.



Places of possible development of cracks in brickwork (red colour designates zones of the main tensile stresses) and location of cracks according to inspection.

Thus, our investigation definitively showed: the reason for opening of cracks is the heterogeneity of the subsoil. This factor was at work for over the 200 years of the building's existence and could not lead to a surge of its deformations in the future. Reappearance of old cracks on the renewed plaster occurred because of the background vibration caused by movement of transport (0,035 m/s^2). The geodetic survey which we conducted in 2002-2005, showed that the settlement development rate of the Stock Exchange does not exceed 3 mm a year which is typical for buildings founded on soft soils exposed to constant dynamic background.

In the long term there are no reasons to expect progressive growth of deformations of the building and if the existing *status quo* is preserved no strengthening of the Stock Exchange will be necessary in the foreseeable future.

As the Dear Reader could notice the experts of "Georeconstruction" Institute prevented a thoughtless and unnecessary subsoil strengthening of the Stock Exchange, to displeasure of many interested parties willing to participate therein. But it is necessary to remember that the motto of true expert, as well as of the true doctor, is always the famous principle "*Primum non nocere*" ("Most importantly, do no harm"). Any strengthening or underpinning is a painful operation. And it should be performed only when absolutely necessary.

The Naval Cathedral of Kronstadt

Once, in the evening, in late May 2009, inside the Cathedral of Kronstadt there was a loud bang, somewhat resembling a shot fired from a battleship's cannon. In arched spans of the transverse walls in the observation gallery through cracks appeared, followed by an outpouring of plaster and noisy falling out of a brick dislodged from the span. Slanting cracks appeared in lateral turrets of the stairwells. This provoked a considerable resonance. As always, a multitude of people sprung forward, aspiring to strengthen or reinforce one thing or another, or to help someone in any way at all. One firm, famous in the field of apartment makeovers went so far as to suggest drilling four boreholes and pumping in concrete – the more the better – so as to save the building on the verge of disaster ("Stop thinking – we need to save it, don't we?").

The Committee for the Preservation of Historic Monuments was instrumental in arresting the chaos. A period of twenty days was allocated to establish the exact cause of trouble (What is happening?) and answer the question: What is to be done? This caused a significant drop in the numbers of aspiring "saviours". Those who not only **wanted to help**, but were actually **able to help** remained and, quite literally "held their ground".

The task proved to be a hard one. The cathedral constructed in 1914 to a design by architect Kosyakov featured a novelty material – reinforced concrete, which was used in constructing the main dome, or, more specifically, its lower hemisphere, visible from the inside. The foundations had been constructed of monolithic concrete, reinforced with steel I-beams. The walls, the vaults and the pilasters were all brickwork. In the 1930s the cathedral underwent reconstruction. The dome part was separated with an overspan, whereby the lower part was turned into a concert hall. In the walls, vaults and dome supports there were multiple cracks which to the onlooker appeared as chaotic. What was the reason for their formation?

Various explanations for the cracks were proposed, such as moistening of walls masonry and subsequent damage during freezing and thawing; subsoil deterioration due to an unknown natural or manmade factor, or rotting away of the timber piles under the footing.

The last version was rather ludicrous. As followed from the report by Kosyakov, when constructing the foundations they had to dispense with the idea of putting in timber piles for the reason of encountering a stratum of firm moraine clay with sand, containing lots of boulders and underlain by a deposit of blue clay. It needs to be said that the site had been thoroughly investigated by our predecessors, who had gone to the pains of drilling 18 boreholes and excavating 6 deep trenches. The mythical "timber piles detached half a meter from the foundation" (?!), was only contained in a report prepared in early 1990 by a company specializing in research of the outer space.

A metallic structure in the foundation body at the depth of 0,6 m (a miniature camera photography inside a borehole).

As became evident to us during site investigation the rocket scientists had encountered timber cofferdam preserved around the building and took it to be timber piles. In reality there were no piles, beams or anything of the sort underneath the foundation. We proved that by drilling twenty small diameter boreholes from which cores of the foundation material and the subsoil were continuously extracted during production. Those cores were sent for laboratory testing whereas in the boreholes themselves we did some filming with miniature cameras. The boreholes were used for dynamic sounding to establish the soil properties *in situ*. That way we were able to obtain maximum information on mechanical characteristics of the foundations and the subsoil at minimum interference. It was proved based on our investigations that the Kosyakov's reports could well be trusted.

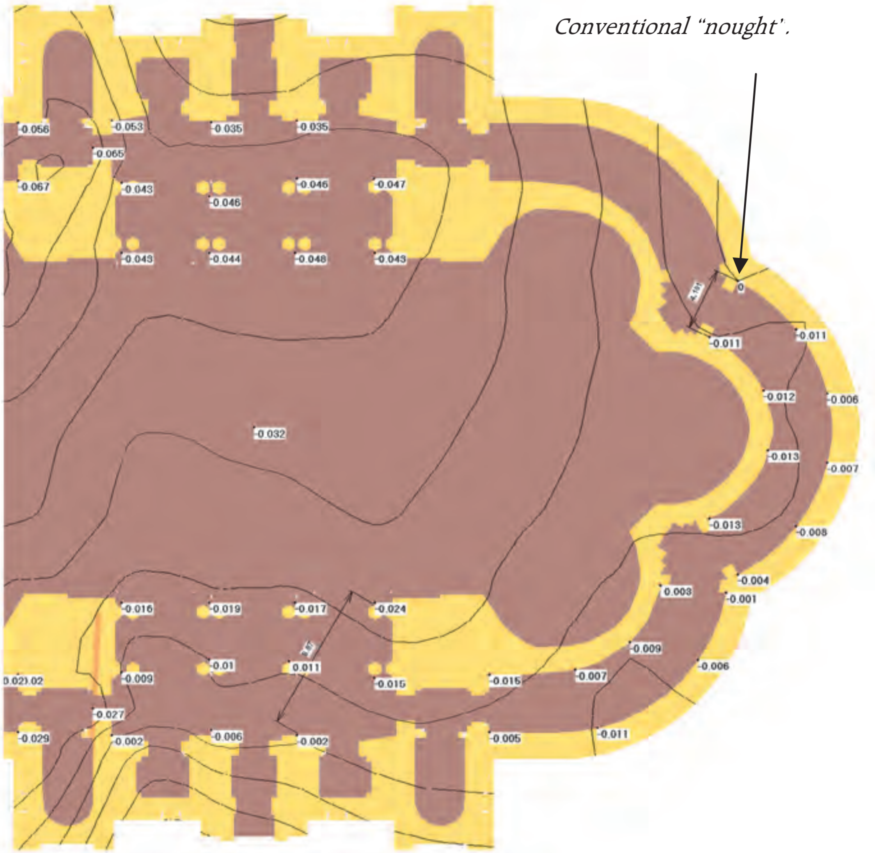
Defects caused by penetration of atmospheric humidity and freezing of the brickwork.

The hypothesis of detrimental impact of freezing and thawing proved quite sound. We calculated that, based on the available ambient temperature statistics for the past one hundred years, every year the masonry froze through by as much as 30 cm, completely thawing every April. In the event of the masonry being damp owing to leaks from the roof or unwaterproofed openings, freezing cannot help but will occasion its decomposition. At the moment of investigation a number of dangerous locations were identified where fragments of the outer "glazing" brickwork course had already detached from the masonry body and were threatening to fall off.

Nevertheless, freeze-thaw destruction failed to explain the overall picture of deformations ongoing in the building, featuring characteristic diagonal cracks in transverse walls, as well as through cracks in the arch overspans and the vaults.

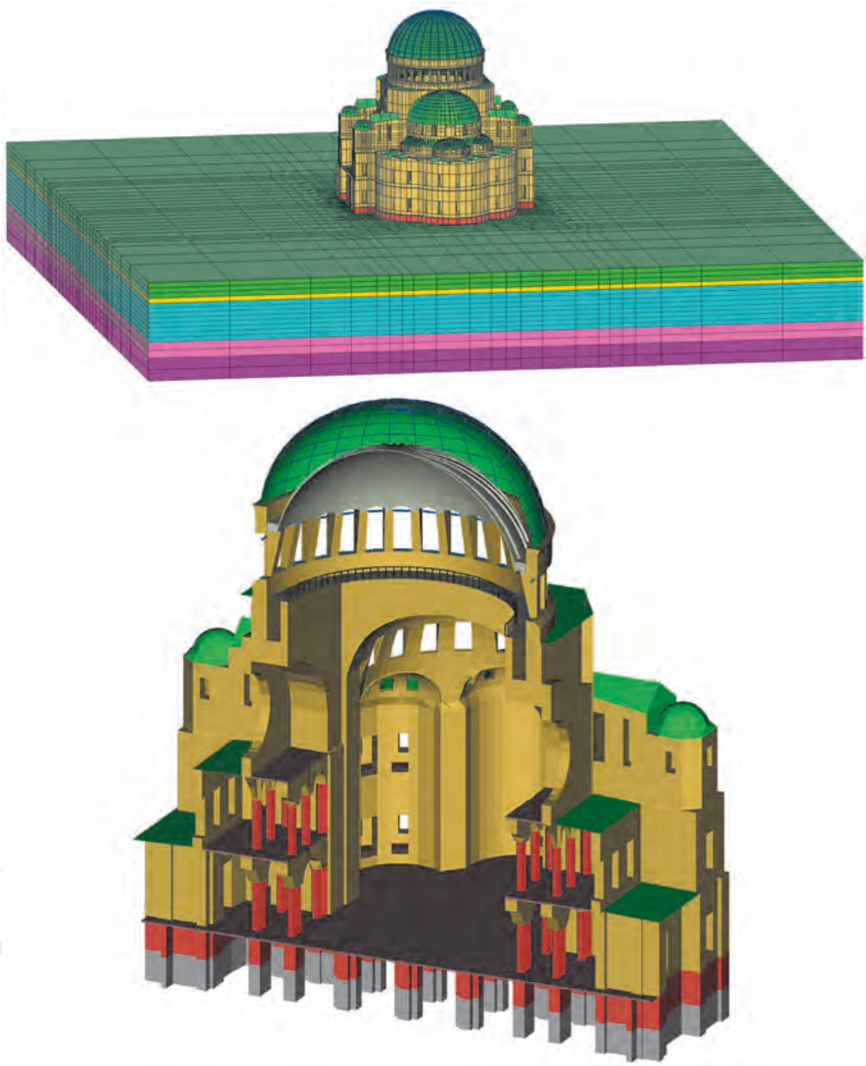
The opinion of our opponents as to all of the cracks having to be attributed to the consequences of moistening and freezing of the masonry did not have a leg to stand on at all. Their main argument was rather curious: according to them, opening of cracks occurred in late May because the cracks contained ice in them, it began to melt and extend (!) in volume. Then (having noticed our great surprise), they changed the concept a little: inside the walls the cold, allegedly, had accumulated, in springtime water started leaking from the roof and found its way into the cracks, where subsequently it began to freeze, *etc., etc.* Elementary calculations of heat conductivity, unfortunately, do not leave any chances for that concept to be viable.

From the very beginning of our investigations we became convinced that the display of the main cracks in the structures of the cathedral is quite typical for a cruciform structure, of which we had surveyed a couple of dozen before. The greatest settlements develop in the most heavily loaded supports – the four massive pylons supporting the dome drum, whereas the smallest settlements will be observed in the external walls. This being so, there should remain "evidence". Inlaid floors of the cathedral were laid after construction of its walls and the arches. As is shown by experience of numerous surveys and monitoring jobs, by this time the building should have obtained 30…50 % of its full settlements. But the rest of settlements should leave its trace as inclinations of floors. Indeed, levelling of floors showed that their levels near heavy pylons were 2 cm lower, than at the external walls. It would be possible to object: the floors were laid unevenly. But the matter is that the inclinations appeared rather natural. It would be unjustified to suspect builders of intended uniform reduction of floor level specifically around the pylons and nowhere else.



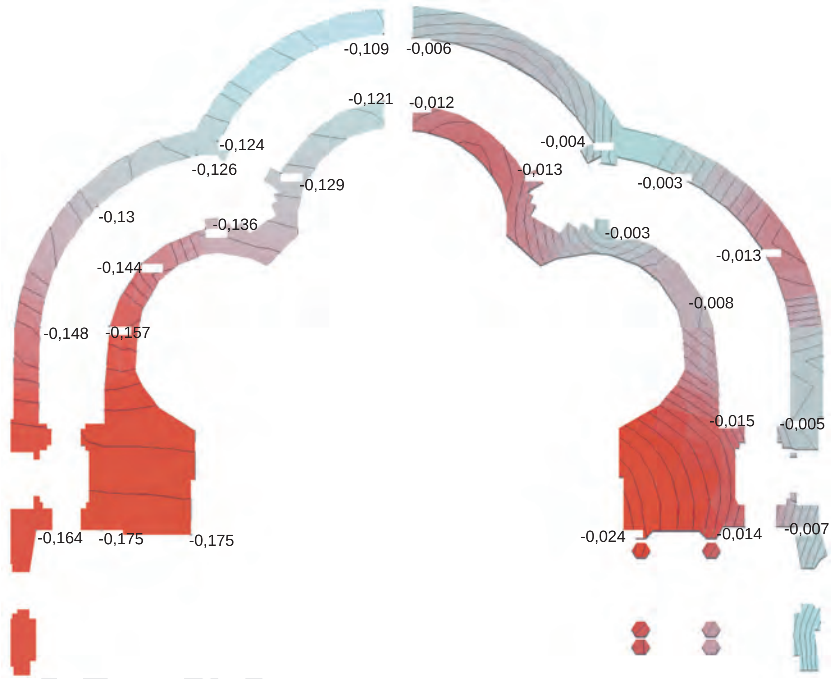
Contours of differential floor levels (m) of the ground floor, based on geodetic levelling.

Another objection as to whether it was realistic for a differential of merely 2 cm to have led to the cracks layout observed in reality demanded a thorough numerical analysis.



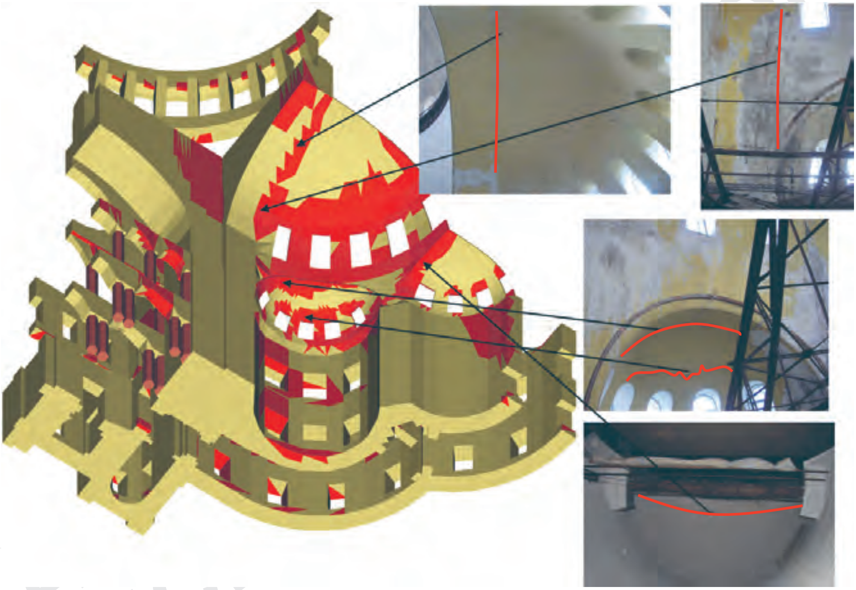
Calculation scheme for the building and the subsoil (general view and cross-section). The subsoil is modelled with spatial viscoplastic elements.

As if from a "Lego" set, the cathedral, with all its constructive elements, was carefully assembled from three-dimensional final elements in *FEM models* software. The entire survey process for that historic building was structured by us so as to receive all necessary parameters for the calculation profile. Durability of brickwork, the class of concrete in the foundations and the lower hemisphere of the main dome were defined, the design of separate joints and elements were specified, properties of subsoil under the foundations were more accurately established.

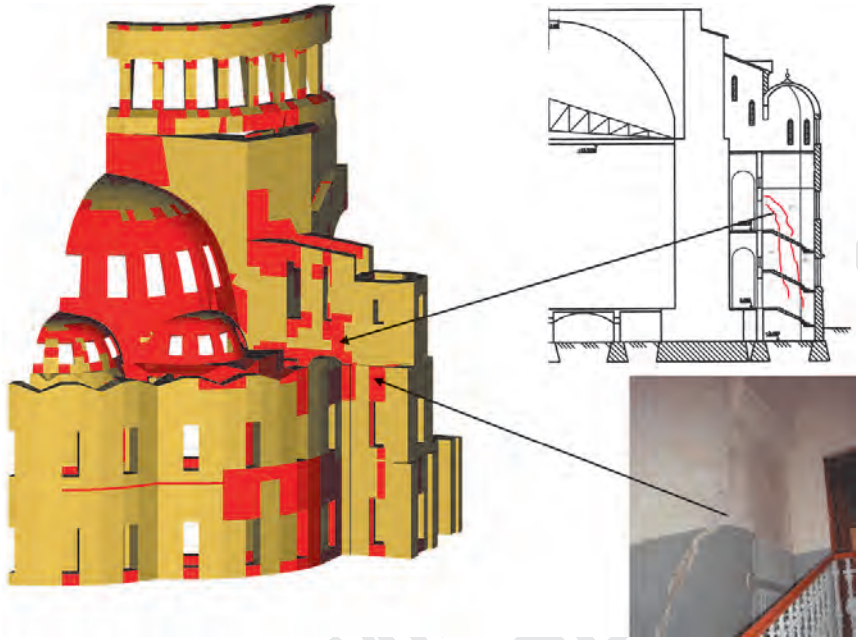


Comparing the character of predicted and measured differential settlement development with account of damage (left and right, respectively). The predicted layout (left) has absolute settlements indicated (m). The measured layout (right) has differential settlement indicated (m), relative to the conventional "nought".

The calculations result turned out very remarkable. It appeared that for the entire time of its existence the settlements of the cathedral reached only 8 cm. To compare – as of today St. Isaac Cathedral has settled by as large as one meter. At small absolute settlements the Kronstadt Cathedral obtained, apparently, a correspondingly small settlement differential: up to 2 cm between the external walls and the dome pylons. However, it was due to those settlement values that tensile stress areas appeared in the cathedral structures. And, as we all know, it is in the tensile zones in brickwork that cracks appear. Comparing the calculation results and the real picture of deformations, their complete coincidence is immediately obvious. Thus, the main perpetrator of cracks in the cathedral had to be its structural layout, having been designed as intolerant to settlement differential.



Places of possible occurrence of cracks (the main normal pressures exceed the maximum tensile strength value). View from bottom up.



Character of crack development (iterative solution of the brittle-elastic problem).

The further investigation should have shown whether there were any signs of deterioration of mechanical properties of the subsoil connected to influence of any natural or technogenic factors. It is not improbable, also, that the mechanism of the cathedral's deformation started even while it was still under construction, and we observed consequences of the "centenary" soil creep. In any case, on the whole, there is currently nothing threatening the existence of the cathedral (if, of course, one manages to appease its overactive "rescuers", always ready to pump anything in its subsoil). There are only local threats. We have already spoken of the possibility for the fragments of the external "glazing" masonry layer to fall off. Another serious problem is the spalling of the brick consoles of columns of the main cylinder supporting the reinforced concrete dome. Here it was necessary to take the obvious strengthening measures immediately. Just how serious the designers and contractors were about the cathedral's restoration we shall we find out in a couple of years. For this purpose it will be enough to look for reappearance of old cracks on the updated interiors.

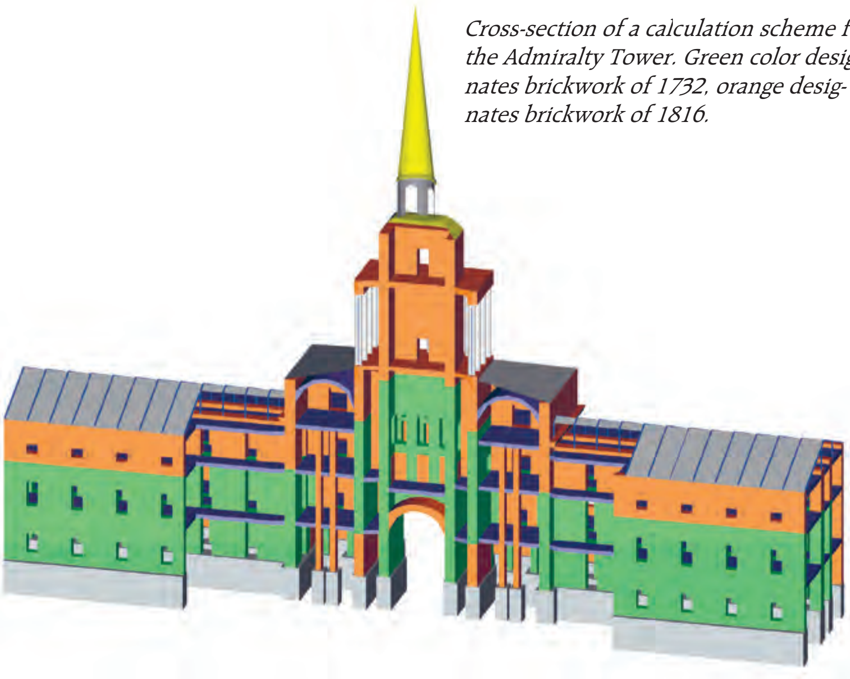
The Admiralty Tower

The tower of the Main Admiralty is another well-known symbol of St.Petersburg. It was constructed in 1734 as designed by I.Korobov and reconstructed in 1811-1823 to the design by architect A.Zakharov. During reconstruction the Korobov's tower was encircled with new façades with a high entrance arch and topped with a colonnade. Recently it became visible that the main front wall was getting separated from the rest of the tower structure by a series of cracks. How is it possible to provide safety of the city's signature building? For this purpose the Committee for the Preservation of Historic Monuments contacted experts from "Georeconstruction" Institute.

Having studied the archival documents, we found out that settlement differentials of Zakharov's walls and Korobov's structures were noticed right after the reconstruction of the tower. The foundations of the tower of both periods of construction were of shallow linear type made of fragmentary limestone slab on lime mortar. Condition survey demonstrated that Zakharov, while disassembling the internal walls preserved Korobov's foundations. But there was no link between the "new" and the "old" foundation structures. The foundations of Korobov's and Zakharov's courtyard façades had been connected with underground brickwork arches (instead of the expected rubblework foundations). The deepest (4,2 m) was the foundation of the front elevation wall. Investigation drilling identified wooden piles under Korobov's foundations. The timber of the piles was found intact and undamaged. Under Zakharov's foundations, despite all our diligence, no piles could be identified: most likely there are no piles, or they were spaced really very sparsely. The body of the foundations, as the video inspection through boreholes showed, had numerous voids owing to washing away of the lime mortar.

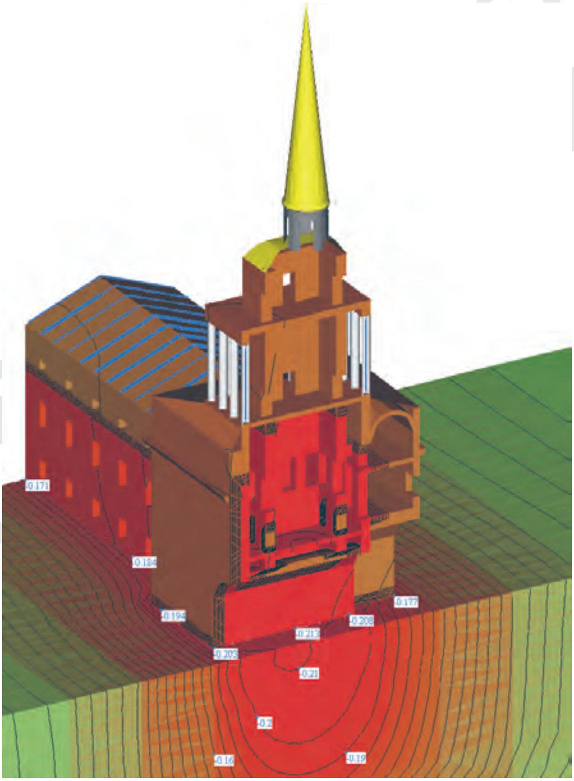
We investigated all possible factors which could influence development of deformations of the tower, including the dynamic impact from the city transport, and the underground trains. It appeared that the dynamic background was by a factor of magnitude below the maximum permissible level.

Perhaps, under the influence of some unknown technogenic factors the subsoil of the tower abruptly suffered deterioration of its conditions? Our investigation showed that the foundations of the tower were constructed on diverse soils – sand fills, natural sands and natural sandy loams. Dynamic sounding performed on these soils showed that despite the heterogeneity, they had mostly dense consistency. Hence, there was no reason why we could expect any adversity to arrive from that direction.



Cross-section of a calculation scheme for the Admiralty Tower. Green color designates brickwork of 1732, orange designates brickwork of 1816.

Revealing the true reasons for deformations and predicting their further development become possible with the help of numerical modelling of the entire "superstructure-foundation-subsoil" system. When modelling by means of our *FEM models* software the objective was to represent as precisely as possible the structures of the building and the history of its construction. First, we determined structural deformations of the Admiralty as it was in 1732, and then we accounted for the change in the computation scheme connected with appearance of additional elements during reconstruction of 1816. According to the results of calculations the total settlement of the tower reaches 20 cm, and the central (Korobov's) part receives greater settlement than the front elevation wall.



Settlements of Zakharov's Admiralty, according to numerical analysis.

This difference is only 3 mm, and is apparently insignificant. But owing to there being no link between the foundations it is enough for development of high tangents of stress in the brickwork of the transverse walls (up to 235 kPa) which lead to formation of cracks. Their opening is aggravated by the overall tilt of the tower towards the front elevation caused by the asymmetric position of the tower relative to the main building. The system of cracks formed a deformation joint which can open under temperature-related influences of seasonal nature. Undoubtedly, the cracks also open due to the so-called centenary creep of the tower's subsoil.

The figure on the left indicates the zones of possible development of inclined cracks, according to the calculations; the photo on the right shows the actual development of cracks on the Admiralty building.

Thus, it was established that the principal cause of development of cracks in the Admiralty Tower is settlement differential, the mechanism of which was launched during construction and reconstruction of the building. No new risk factors were revealed. Thus we could prescribe the building the conservative "treatment": strengthening of the foundation masonry by means of injection and restoration of spatial rigidity of the building by connecting its front elevation wall with internal walls of the tower.

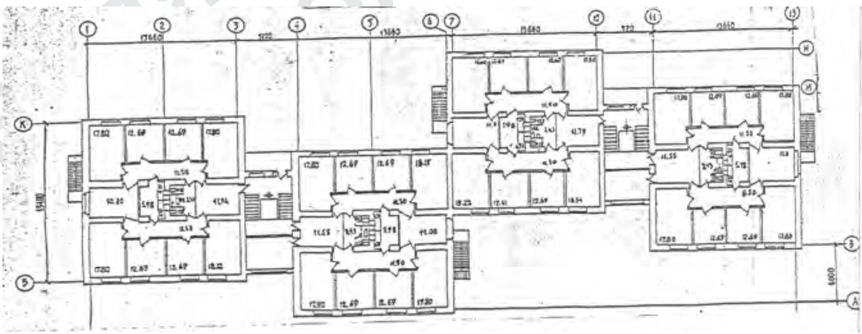
Collapse of an apartment house on Dvinskaya Street in St. Petersburg

Dear Reader, when on the pages of this book we spoke about the risk for safety of the building, what we meant was the risk of its damage. Collapse of buildings is a risk of a much higher level of magnitude. In the risk theory developed in the West today it is considered impermissible if owing to an oversight it becomes necessary to evacuate residents from more than one building out of one hundred thousand a year. It is our very well educated guess that in our country this risk is by one or even two orders of magnitude greater. The cause is non-professionalism which, alas, has always been there but these days, when design is being done by completely unprepared individuals, it has become blatantly belligerent. Collapsing buildings have the power for some time to sober up the lovers of amateur "instinctive" design and "random" construction. But time passes, and the lessons are forgotten. Therefore, it would be not altogether out of place to remind ourselves of one of such cases.

A huge resonance in St. Petersburg and in the entire country was caused by the catastrophic collapse of an apartment house on Dvinskaya Street in 2002.

Catastrophic collapse of an apartment house on Dvinskaya Street, 8.

A brick building constructed in the 1970-s, without any cracks opening in excess of 3…4 mm, collapsed and crumbled into dust over some 40-50 minutes! When authors of this book were involved by Gosstroy to investigate the reasons for the accident, they identified an entire array of mistakes made during design and construction of that building.



A typical floor plan of Dvinskaya Street, 8.

First of all, the 9-storeyed brick building, compounded of four blocks, did not have enough spatial rigidity. The walls were cut by many windows and doorways. No rigid staircases were provided. The blocks of stairs were positioned in the weakest places of the building – in the links between the blocks. For such building to exist safely it had to have been given a very reliable, almost settlement-free foundation. In the geological conditions of the site, where the layer of fills was underlain by peaty soils, with soft clay underneath, the only possible type of foundation could only have been a piled one. But at the time of its construction, driven reinforced concrete piles were only beginning to be used for residential buildings and were difficult to find. The designers invented an original concept to "legalize" their shallow foundation option. They did not take into consideration the site investigation report prepared in 1969, but instead inscribed the drawing with the following note: "Foundations were calculated as per the instruction of the institute's Senior Engineer (the following values were assumed: $\varphi_н = 20°$, $g = 2$ t/m^3 at $E = 100$ kg/cm^2)". Needless to say, these values certainly had nothing to do with the actual investigation results. Such foundation solution mismatched the geological conditions of the site, and was not adequate in relation to the typical box-shaped structural layout incapable of assuming settlement differentials. To these should be added the following "mere trifles": the foundations were constructed with the lowest quality imaginable, the cushions (obviously, with a view of economy in mind) were positioned at big intervals, in some places the foundations were compounded of wall blocks, the reinforced waling along the undercut of the foundations was of poor quality, and the brickwork reinforcement in the walls stipulated by the design was missing altogether.

In the given situation, instead of the question as to the reasons of collapse another question should have been asked: how on earth had the building managed to stay in one piece for 30 years?!

The building science does not have the answer to this question. At all times and for all peoples its purpose was to provide safety and reliability of buildings, instead of experimenting on living people: "will the house collapse or not if we violate the laws of construction mechanics?".

And the sequence of events was as follows. On May 22, 2002 the residents of Dvinskaya 8 again discovered their basement flooded. On May 30 and 31 water discharge was noticed on the adjacent lawn and near the building. On June 3, when the maintenance crew had arrived, there was no water in the basement. Workers started to repair the pipe that fed water into the building. For this purpose they dug out a small trench 1,4 m deep with plan dimensions of 1.0 by 1.5 m. Directly after the digging they heard a loud bang from the basement area, and there appeared a crack in the building. Sand started pouring into their trench and the maintenance crew was evacuated. A fire broke out in the building. The southern section began to tilt intensively in the southerly direction forming a split between the adjacent sections. In 45 minutes the entire southern section collapsed.

Fortunately, the residents had been all evacuated through a faultless action of a rescue team of the Ministry for Emergency Measures. The southern section had developed settlement of about 0,5 m. The volume of the settlement wedge totalled approximately 50 m^3.

So, the collapse of the house was, apparently, associated with the broken down water supply systems. But was it really the main reason? It is necessary to point out that some “experts” like to explain away deformations of buildings in our city by the fact that someone had opened some “tap” (or a water pipe burst), and so the settlements started. But if you closely examine the situation, it will appear that it isn't just some “tap” which is the reason.

For the given situation, when there was a simultaneous action of several negative factors (designer's mistakes, contractor's pig-headedness, negligence on the part of the maintenance organization), an authentic analysis of the reasons for damage is only possible with account of interaction between the building's subsoil, foundation and superstructure. We performed a series of numerical analyses using our program complex *FEM models*.

To model the brickwork we used plate elements assuming fragile-elastic behaviour of the material.

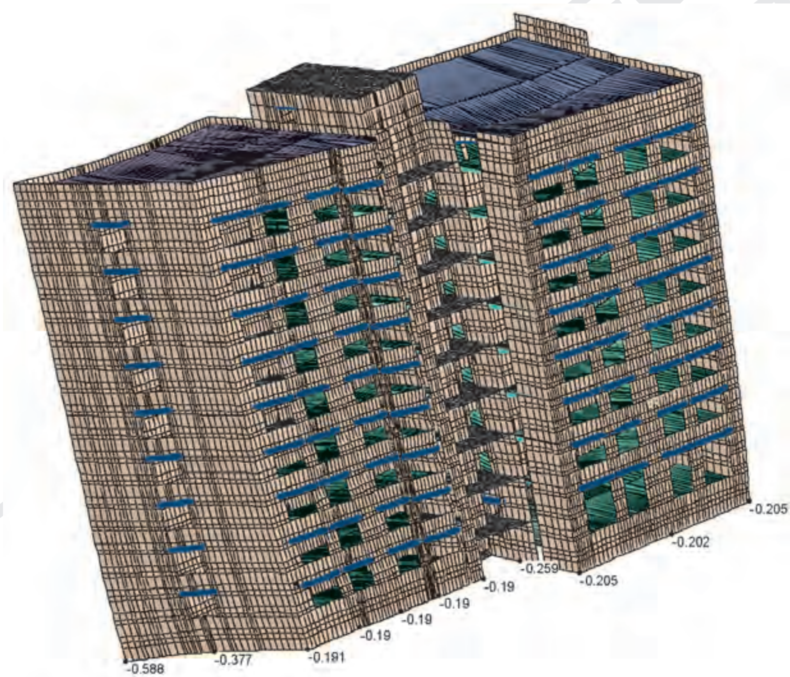
The characteristic outlook of settlement-related cracks for buildings with reinforced concrete crosspieces: left – according to numerical analyses; right – on the building adjoining the accident site (Dvinskaya 8, Building 2).

Soil-structure interaction analysis allowed not only to calculate the absolute settlement values, but also to reveal the spatial character of settlement differential distribution. According to the calculation, the greatest settlements (up to 26 cm) had been accumulated on the internal walls of the building, the maximum differential (0,003) observed on the emergency section.

Such differential leads to formation of a series of vertical cracks in the walls, characteristic for buildings with reinforced concrete crosspieces. Unlike older buildings with wedged brickwork crosspieces, to which inclined settlement-related cracks are specific, presence of more rigid reinforced concrete crosspieces leads to deformations of window blocks happening in the form of their turning in place as a rigid whole. The cracks of this character were obvious in the preserved blocks of the fallen building, as well as in the structures of the adjacent house constructed to the same design. The cracks divide the walls into free standing separate columns, depriving the building of spatial rigidity. At first the building is saved by the fact that its settlements profile looks like a “hammock”.

The columns get pressed together and the building stands. But should there be interference of any factor, capable of lowering the edge of the "hammock", there will be nothing anymore to keep such structure from collapse.

The digging of the trench by the maintenance team in order to repair the water pipe was the last straw which led to the subsoil getting squeezed from under the foundations. The subsoil had reached its limit state due to mistakes made during design and construction. Appearance and quick disappearance of water in the basement and near the building prior to collapse was indicative of the fact that the entire subsoil had been washed through by industrial water which had generated suffusion courses enabling it to escape into sewer collectors or into the backfilled Seldianoy Canal located nearby.



Emergency deformations of the building according to soil-structure interaction calculations.

All these maintenance-related issues would have had no significance if it had not been for the mistakes made by the designers. They should have insisted either on using piled foundations which, given the circumstances of the time and the place, were not easy to get hold of, or on getting assigned a standard design of higher spatial rigidity.

The accident at Dvinskaya Street is an example of how each participant of the building process (the designer, the builder, the maintenance organization) robbed the house of a fragment of its durability and of what remained of its durability as a result.

Chapter 23. Walk Two. For art lovers. The Palace, the Theatre, the Concert hall.

The projects about which we cannot wait to tell you, Dear Reader, are remarkable because of the fact that their construction went very quickly and ended successfully. Their success was guaranteed by high professionalism of the client, the designers and the contractors. Their mutual relations were dominated essentially by their being united in the common cause. (In the Russian language, quite aptly, "the common cause" and "the common business" have very different, perhaps even contradictory meanings.) No one tried to pass the buck, and everyone was quick in dealing with matters concerning the sphere of their competence entirely.

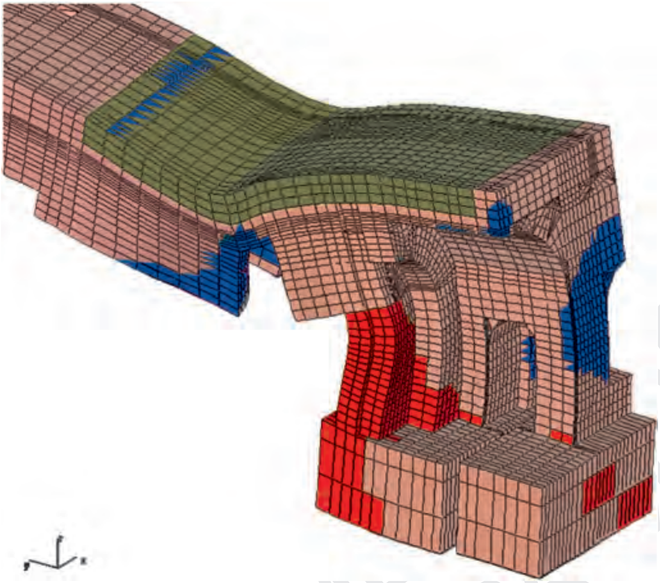
The Konstantinovsky Palace

In the year 2000 the Committee for the Preservation of Historic Monuments commissioned "Georeconstruction" Institute to execute condition survey of the Konstantinovsky Palace located in the St. Petersburg suburb of Strelna. The eyes of specialists were greeted with a truly horrifying view.

Konstantinovsky Palace in the St.Petersburg suburb of Strelna, 2000.

The loggias and the grottoes forming the retaining structures, keeping Konstantinovsky Palace in place on the very edge of the slope (the ancient shore of the Baltic Sea) were on the verge of full destruction. The powerful external wall separating a suite of cellars from the Lower Park, in many places collapsed, through the breakages in it one could look out into the park. The ruins were bathed in the waters running down the slope from the top terrace. Later, when the frosts hit the land, ice stalactites of odd shapes were formed. The ground was washed out from under the cross-section walls – it was really removed by the streams of precipitation water discharge flowing through the retaining structures. Some walls had tumbled down and lay on one side. The calculations showed: unless urgent measures were taken to rescue the retaining structure, it would collapse completely in the nearest future. After this the doom of loss will brush upon the cheek of the palace itself, standing on the edge of a slope without protection.

Cross-section walls of the retaining structure, forming a terrace in front of the palace, as witnessed by condition surveyors.



"This is numerical analysis speaking": "If you fail to act soon, taking urgent measures to save the building, the retaining structures will be destroyed, and then the shadow of death will hover above the palace itself. Over?!".

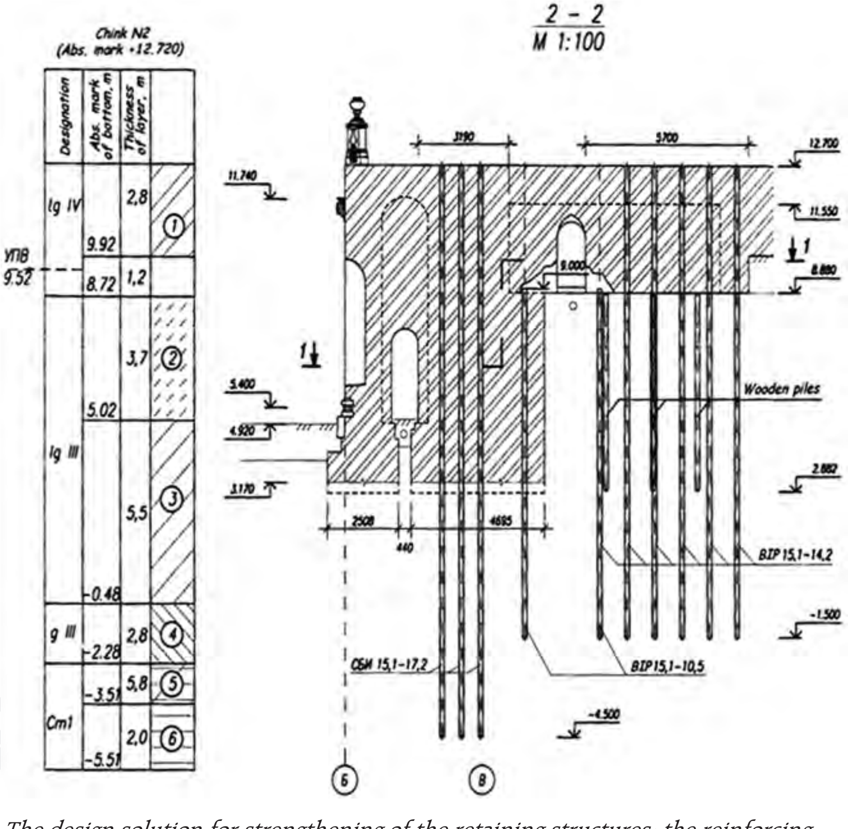
Destinies of buildings – like destinies of people. Some are lucky, the others have been bypassed by Fate. Konstantinovsky Palace was the typical loser from day one. It was conceived by Peter the First, and started by the architect Nicola Michetti, as the state residence, a token of Russia's growing strength on the Baltic. But something had gone wrong with the fountains. And Peter lost his interest in Strelna, presented the palace to his daughter Elizabeth, and himself took a much greater interest in the-not-so-far-away Peterhof. The unfinished palace stood vacant until Elizabeth's accession. Francesco Bartolomeo Rastrelli completed the palace and it was ready to receive the imperial court. But, again, this was not to happen. Catherine the great of Russia perhaps did not even know about the palace, and Emperor Paul presented it to his son Konstantin. That's where the palace got its name from. The architect Andrey Voronikhin in 1803 completely updated interiors of the palace, but a fire broke out the same year and destroyed all the furniture. The palace was then completely renovated by the architect Luigi Rusca. However, already in 1848, the architect Andrei Stakenschneider found the palace in the state of decay and made himself busy with its reconstruction. But after the Russian revolution, desolation again awaited the building. During the Second World War the palace was really strongly damaged. In post-war time only the facades and two halls were restored. The palace housed the Arctic School for some time.

After departure of the School dilapidation started to gain strength again. Precipitation water discharge system passing through the brickwork of the retaining structure got locked in the drains as they had been clogged with rubbish. Whilst being trapped inside, precipitation water froze during the winter and damaged the brickwork. It was necessary to radically change the system of water removal, having provided drainage away from the palace, bypassing the loggias and the grottoes. But how was it possible to rescue the walls of the loggias, the grottoes and the suites of cellars which comprised the retaining structures for the monument if the supporting wooden piles had decayed, and the brickwork foundations had turned to dust?

It was requisite to solve one more problem, realization of which was necessary for the new life of the palace: to organize the main entrance through the grottoes of the Lower Park. The visitors, arriving by sea, and then proceeding along the canal towards the palace, should have been met inside a spacious lobby.

So it had been conceived by Peter the Great in the Peterhof Palace. Here there were only the low cellars to start with.

We are proud to have been able to conceive of the solution capable of tackling all problems. We used drilling rigs first to drive out upon the terrace in front of the palace and drill the brickwork of the retaining structure with small diameter boreholes – 43 mm. Through these apertures careful mortar injection into brickwork was carried out. After that, the apertures were widened to the diameter of 150 mm. The boreholes went completely through the brickwork reaching strong deposits of Wendian clays. Thus inside the historic masonry a reliable reinforced concrete skeleton was formed, turning into piles at the lower levels. On the surface of the terrace the heads of reinforced concrete piles were incorporated into a monolithic slab upon which the system of thermal snow removal and a granite pavement were arranged.



The design solution for strengthening of the retaining structures: the reinforcing piles pierce the retaining structure and transfer the load onto stronger soil strata.

The most fascinating show for a geotechnical engineer was the view of the cross-section walls of the retaining structure which were dug by 1,5 m for placing the engineering services. Massive brick walls were “hovering” on thin strengthening piles. It was possible to glance under the foundation footing and to see rows of piles, 50 m on each side, under the historic walls.

All walls of the retaining structure got suspended on thin strengthening piles.
It was possible to look at the undercut of the foundation footing.

In the area of the main entrance it was required to provide a 5 meter wider and deeper space (which is a lot, by the way) for which purpose a retaining wall of bored piles had been designed.

The strengthening was executed so reliably that no block of brickwork received any deformations over the entire period of works.

The realized design solutions based on the advanced methods of calculation, as well as the scientific support of strengthening works for the foundation and the subsoil of the Konstantinovsky Palace merited a high grade. The scientific supervisor of the project, professor V.M. Ulitsky received the State Prize in the Field of Science and Technology. On this reconstruction project which was realized extremely fast, the union of design and science was demonstrated, which ensured everyone's success.

Konstantinovsky Palace after reconstruction.

Kamennoostrovsky Theatre

Even more geotechnically courageous undertaking was realized on Kamenny (Stone) Island in St. Petersburg commissioned by the Committee for Preservation of Historic Monuments. This was the overhaul of an old wooden theatre to be used as the Second House of the Bolshoy Tovstonogov Drama Theatre.

Kamennoostrovsky Wooden Theatre. How was it possible to transform it into a co-temporary thespian project, having kept the authenticity of the monument? The answer: all things new had to be hidden underground. A complicated geotechnical task indeed.

The theatre building was constructed by architect Smaragd Shustov in 1828 as a temporary stage for performances of the Imperial Theatre of Opera and Ballet. The matter was that they had not had enough time to finish the major overhaul of the Big Stone Theatre (which was later to become St. Petersburg Conservatoire). He works, as usual in Russia, got a little delayed. That was how Mr. Shustov came to design a temporary wooden theatre which cost only 40 thousand roubles in gold and had to be constructed over the time of 40 days. However, in practice it was twice as expensive for the royal coffers (that too is quite a frequent phenomenon in Russia). But the resulting theatre was excellent, of rare lightness and grace. It was a sheer pity to have it dismantled. About fifteen years later the well-known theatrical architect *Albert Kavos* was charged with its capital reconstruction (that time Shustov was not invited; obviously as having previously doubled the cost and not having been forgiven). Kavos promised that the theatre would stay intact for another 50 years. But it stayed intact for another 180. The wooden theatre survived the fires of the October Revolution. It survived also the bitterly cold winters of the German Siege of Leningrad, no one dared burn such an elegant thing. Today there are only two wooden theatre buildings in Russia. Till the 1930s, the Kamennoostrovsky Theatre had been used as a warehouse, then it was repaired and placed in the hands of the Television Theatre, and later of the studio of dance school.

New life of the Kamennoostrovsky Theatre began in 2006 when the President of the Russian Federation in connection with the anniversary of the famed Russian actor Kirill Lavrov moved it into the ownership of the Bolshoy Drama Theatre (the BDT).

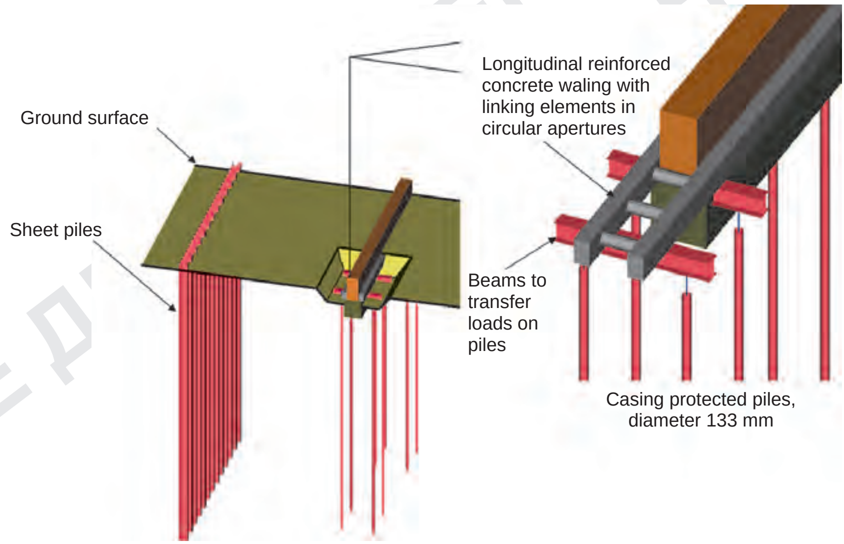
A Modern theatre is just like an iceberg: its visible and accessible part is the smaller one, the bigger one being hidden from spectators.

The invisible part houses stage equipment, warehouses for "stage design" and the props, the extensive air duct facilities silently supplying air to the stalls, *etc.* It also houses dressing rooms, maintenance and ancillary premises.

Over the 180 years life of people changed also: there will be no footman holding your fur coat waiting for you near the theatre once the performance has finished, we now need wardrobes, and comfortable toilets. And without a cafe a theatre now is certainly incomplete. Nothing of the kind could be found in the historic building.

How was it possible to reconcile the seemingly unsurpassable contradiction: to preserve the original historic building of the theatre making it modern at the same time?

Here again we are aided by geotechnical engineering. Everything new can go underground. This was the reconstruction strategy adopted by the Committee for the Preservation of Historic Monuments. It was not by chance that the senior designer's function was entrusted to "Georeconstruction" Institute known, first of all, for its geotechnical design prowess.



Sheet piling cofferdam, bored piles and the element of wall-to-pile load transfer.

The historic building was reseated onto "Titan" piles 18 m long. These piles are unique because during the works they are settlement-free. To transfer the loads from the existing walls onto piles an artful transfer unit was invented. On each wall side reinforced concrete beams connected to each other through apertures preliminarily drilled in the socle were provided. Under this waling a small window was arranged on each side, into which a steel beam was inserted, itself resting on a pair of vertical piles. The beams were jacked and the loading from the walls was transferred to the piles. After that it was possible to disassemble the no longer necessary old foundations.

Reinforcement for the concrete waling.

Insertion of beams under the waling and transfer of load to the pile.

The underground space under the theatre exceeded the footprint of the historic building. Strengthened steel sheet piles were driven alongside the border of the underground scope. In the meantime old rubblework foundations were disassembled and the ground excavated to make room for a reinforced concrete raft at the relative level of minus 2,0 m. This raft was the future overspan above the underground floor. All works were organized so as to exclude welding – no fire in a wooden building, gents!

All loading is transferred to piles. The old foundations are removed (the photo also illustrates the point of view of a geotechnical engineer on architecture).

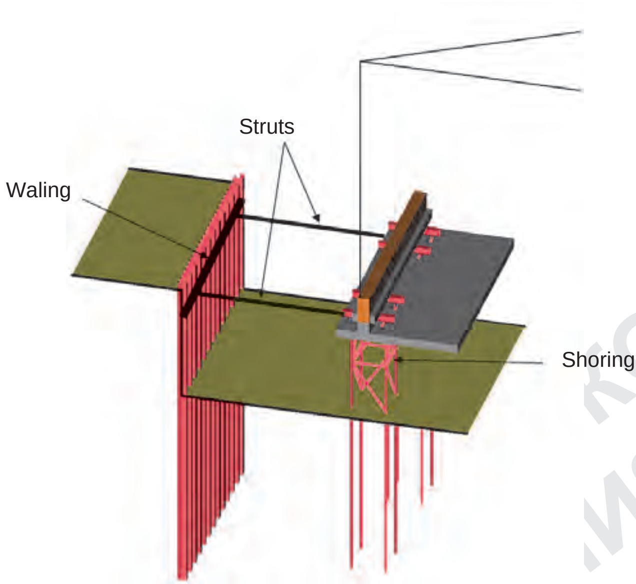
The Committee for Preservation of Historic Monuments skilfully organized work of a remarkable team of professionals.

Works on the underground part were conducted by company "Geoizol". Such level of professionalism, such culture of works implementation could be met on very few sites, there you can stand on us! And all this – in the most complicated constrained conditions. One could always visit the site wearing smart shoes not being wary of prematurely losing them.

Placement of the top slab.

Restoration of the timber structures conducted under scientific management of Research Institute "Spetsprojectrestavratsiya" by the firm "Kraski Goroda", and at the final stage by the firm "Intarsiya" merits a separate story, just like the unique staging technology proposed by the firm "Theatrical and Staging Workshops". Here we shall also dwell in a bit more detail on the geotechnical aspect of the project.

So, the historic building is now comfortably and reliably resting on piles united with a hard disk – a slab at the level of minus 2,0 m, and the contour of the underground structure is surrounded with sheet piles. The sheet piles are supported by a waling beam; between the beam and the reinforced concrete slab under the theatre there are struts constructed of 800 mm diameter steel tubes to rule out horizontal displacement of the sheet piles.



Excavation to the design foundation level with installation of shoring.

It is known possible to start excavating the underground volume. It is here that the most interesting part of the story begins. In Petersburg builders until recently had not dealt with excavation of soft clay (if you remember, we compared it to thick swill, or thin jelly). This material gets heaped up into a lorry and starts spilling over the brim of the skip by the time the lorry reaches the site gate. Even tips and recycle centres refuse to accept it. They say: before we can accept this soil please remove the water from it. And how on earth is it possible to remove it if 10 thousand years of the soil being undisturbed have elapsed and it is still there?

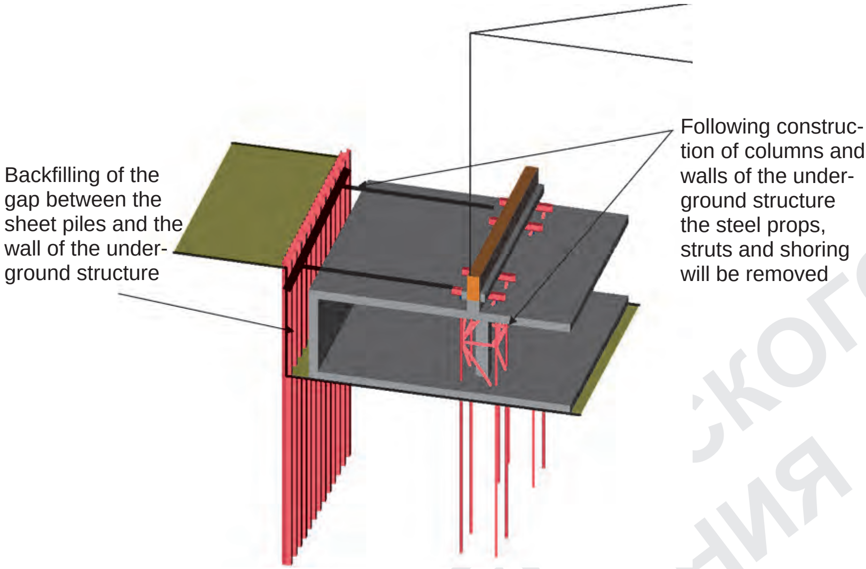
If this soil is remoulded, all construction machinery will simply sink in it. That is why the excavation works were conducted with greatest care (by the contractor called "CZSK"). The soil between the sheet piles and the theatre building was being removed by miniature light-duty excavators. But under the footprint of the a building there was a veritable wood of piles. There was no room for manoeuvre even for the miniature machines. Therefore even in the 21^{st} century one cannot build things without manual labour, of which the tools have still remained the spade and the wheelbarrow.

Struts constructed of steel pipes have been placed between the top slab and the sheet pile wall. Excavation under and around the theatre is underway.

In the process of excavation, pile supports were united by means of steel struts to through spatial columns (for reliable maintenance of their stability).

At the depth of 6,7 m the reinforced concrete bottom slab was cast, followed by external and internal walls and columns propping the slab at the level of minus 2,0 m. Now it was possible to cut the piles within the confines of the underground floor. They were no longer necessary: the bottom slab works as the pilecap, whereas the weight of the historic building is transferred to it through a system of columns and walls of the underground floor.

In process of excavation piles were tied with a system of shoring. At the depth of 6 m the bottom slab was cast. This is the first case of the top-down method being used in St. Petersburg (General Designer – "Georeconstruction", Earthworks Contractor – "Geoizol").



Concreting of walls and columns of the underground structure.

After the struts were removed, and the reinforced concrete span covered the entire contour of the underground structure, the awe-inspiring sensation (which must have been felt by more or less any not indifferent person who visited that site) largely disappeared. A contemporary theatre-goer will now think that everything must have been precisely like that: a cosy wooden theatre with fine acoustics – above, and a convenient underground floor – below. The authors of the design had wanted it to be exactly like that: the new things should have become imperceptible, not distorting perception of the historic building. But let us return to construction.

The sheet piles were perfectly instrumental as the waterproofing barrier. There was no ground water fluctuation in the vicinity, apart from the regular seasonal changes.

All works on the site were accompanied by the strictest instrumentation-aided monitoring. Parameters of vibration generated by construction machinery, settlements and tilts of the actual historic theatre building, and sheet piles movements were carefully and continuously monitored. The special attention was given to the safety of the existing building – the former summer residence of Baron Kleinmichel. All necessary measures were taken even at the design stage of the project. The permissible settlement level had been set as only 1 cm – three times more strict, than allowed by the codes. Actually, by the moment of the ground works completion, settlements of the summer residence house had not exceeded 6 mm. Such a result is the best token of what you can achieve if you conduct your works in a civilized manner. It meets even the strictest requirements of the British Standards.

The technique we used to arrange the underground space on the project in question is similar to the well-known *top-down* method which we presented in Chapter 6. Ours was the first example of its successful realization in ground conditions of St. Petersburg. In this respect it is hard to over-estimate the experience we obtained. Approbation and adaptation of the *top-down* method on that site cleared the perspective for its future successful application in the underground space of our city.

Highlight of the project became our modification of the top-down method, whereby restoration proceeded upwards and construction of the new structure – downwards.

We invite you, Dear Reader to visit this unique building.

Mariinsky-3

And right now we would like to briefly introduce you to one more theatre, to be precise, the Mariinsky Theatre Concert Hall, or as we named it, “Project Mariinsky-3”.

Its construction commenced almost simultaneously with the notorious Mariinsky-2 when reduced excavation works were carried out there. Mariinsky-3, however, was finished over a year. For seven years already a concert hall capable of seating half a thousand people and boasting amazingly good acoustics has been open to music lovers (whereas Mariinsky-2 has only just been completed).

The success was achieved through harmonious cooperation of the designers and the contractors, skilfully organized by Mariinsky Theatre management team.

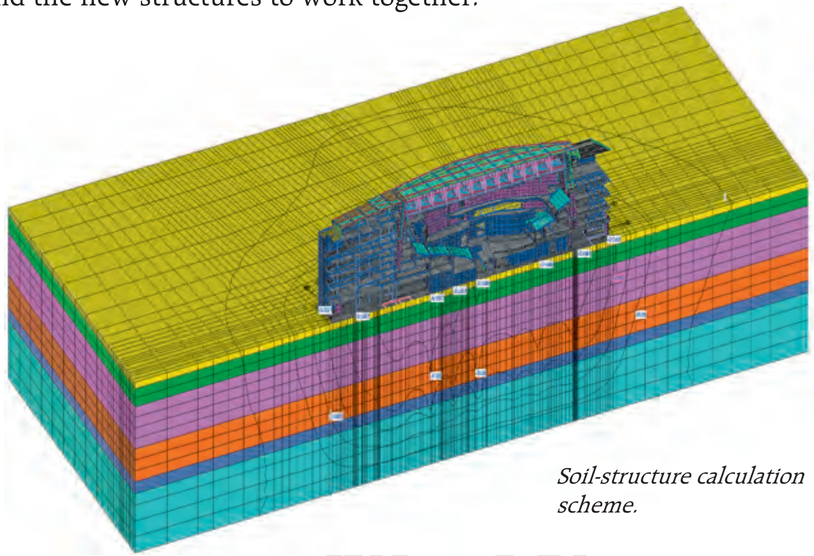
When stage-design workshops had burned down, Valery Gergiev made a decision to construct a performance venue in their place, carefully preserving historic facades overlooking Pisareva Street, constructed to the historic design by architect Shreder. The architecture of the project was entrusted to Xavier Fabre the author of a number of successful thespian buildings in France, and the structural design – to "Georeconstruction" Institute. The acoustics of the concert hall was developed by Yasuhisa Toyota the world-famous acoustician from Japan. The parts of the general designer and the general contractor were played by the company "Neviss-Complex".

Xavier Fabre had a difficult architectural problem on his hands: to arrange an advanced concert hall in the Procrustean bed of the former stage design workshops.

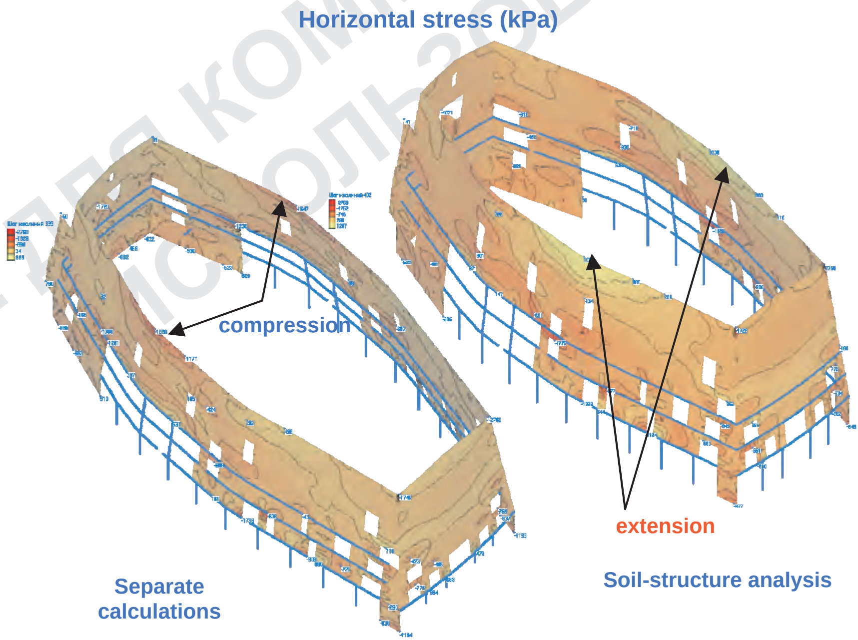
Mariinsky Theatre Concert Hall: preserved historic outlook and a view of the interior.

The building turned out very compact, and the hall – spacious. It resembles an oval bowl fit inside an extended hexagon of external walls. Three of the walls, forming the Greek (or the Russian) letter "П" and overlooking Pisareva Street are historic, as designed by architect Shreder, and it was those walls whose historic architectural appearance was absolutely and totally preserved. Specialists from

"Georeconstruction" had to resolve the challenge of getting the old and the new structures to work together.



Soil-structure calculation scheme.



Comparison of the separate calculations ("on a rigid plate") and the soil-structure analysis.

It could be done only taking into account the interaction between the building's superstructure, foundations and subsoil. "Georeconstruction" specializes in this area, being arguably the most complex in design work.

In professional circles the Mariinsky Concert Hall became a classic reference book example of the requisite necessity of such calculations. Indeed, if you had tried to be "old school" about it, *i.e.* if you had put your calculation scheme on "a rigid plate", or even if you had tried to represent the subsoil with springs, you would still have experienced a complete fiasco. The values of real loads in the building's structures were directly the opposite. Where compression was expected, there we had extension, and where we might have thought there would be extension, there actually was compression. The realistic view of loads can only be obtained by soil-structure interaction calculations. And there the subsoil should best be represented in non-linear setting. Today this is achievable with the software complex *FEM models*, developed and used by "Georeconstruction" Institute.

It is pleasant that our achievements in the field of soil-structure calculations have been noted and recognized by the international geotechnical community. Today the authors of this Guidebook head the Technical Committee 207 "Soil-Structure Interaction and Retaining Walls" of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE).

Chapter 24. Walk Three. For lovers of the extreme. The Highest and the Deepest.

Not all projects of our third walk have yet been born. Nevertheless, for their birth to actually happen a lot has already been undertaken.

The Neva Skyscraper

We shall not name the address and the exact name for this project. Any associations as to the identity thereof should be thought of as casual and fictitious. We shall tell you, as it were, about one of the 400-meter tall buildings, which, allegedly, will be built in our city.

St. Petersburg has a meritorious history of high-rise construction. The brave Domenico Trezzini in 1710 built the bell tower of the Peter and Paul Fortress with the spire reaching 124 m. To this day this is the highest architectural structure in our city. Even Francesco Bartolomeo Rastrelli could not surpass this high-rise record. The bell tower of the Smolny Cathedral should have risen above the belfry of Ivan the Great together with the Kremlin Hill in Moscow (counting from the water level in the Moscow River), just as the empress Elizabeth had wanted. But – "*owing to weakness of soils*" – they could not help but abandon that daring enterprise. With some annoying instances of misdemeanour, in the centre of St. Petersburg everyone still "tries" to observe the imperial decree: civil buildings should not rise above the eaves of the Winter Palace.

Existence of this decree was made known to "Singer" after they had purchased, in the year 1900, for one million gold roubles, a small site located at the corner of Nevsky Avenue and Ekaterininsky Canal. Their dream of a brickwork skyscraper in Chicago style was fated not ever to take shape. It was possible only to arrange a smallish domelike turret which "*beareth the name of Singer*". Today very few people in St. Petersburg know that the contemporary St.Petersburg "Book House" is in fact a truncated American skyscraper. In its brickwork is immured a skeleton of structural steel, exactly like in the early American skyscrapers. We discovered it when doing a condition survey of the "Book House". It needs to be said, though, that the idea of steel-work-brickwork skyscrapers was quickly abandoned for the reason that such buildings were extremely heavy. High-rise construction in North America went along the way of using steel skeleton and reinforced concrete structures instead.

The "Book House" at the corner of Nevsky Avenue and Griboedov Canal, a would-be skyscraper.

It is amusing to see the idea, which became outdated already in the 19-th century, suddenly spring back to life in our city in the beginning of the 21 century. Our high-rise estates in bedroom communities around central St. Petersburg feature lots of 25-storeyed brickwork skyscrapers. In essence, they are the same as 9-storeyed log huts. Something which is certainly possible to build, but what for?!

Steelwork inside the masonry of the "Book House".

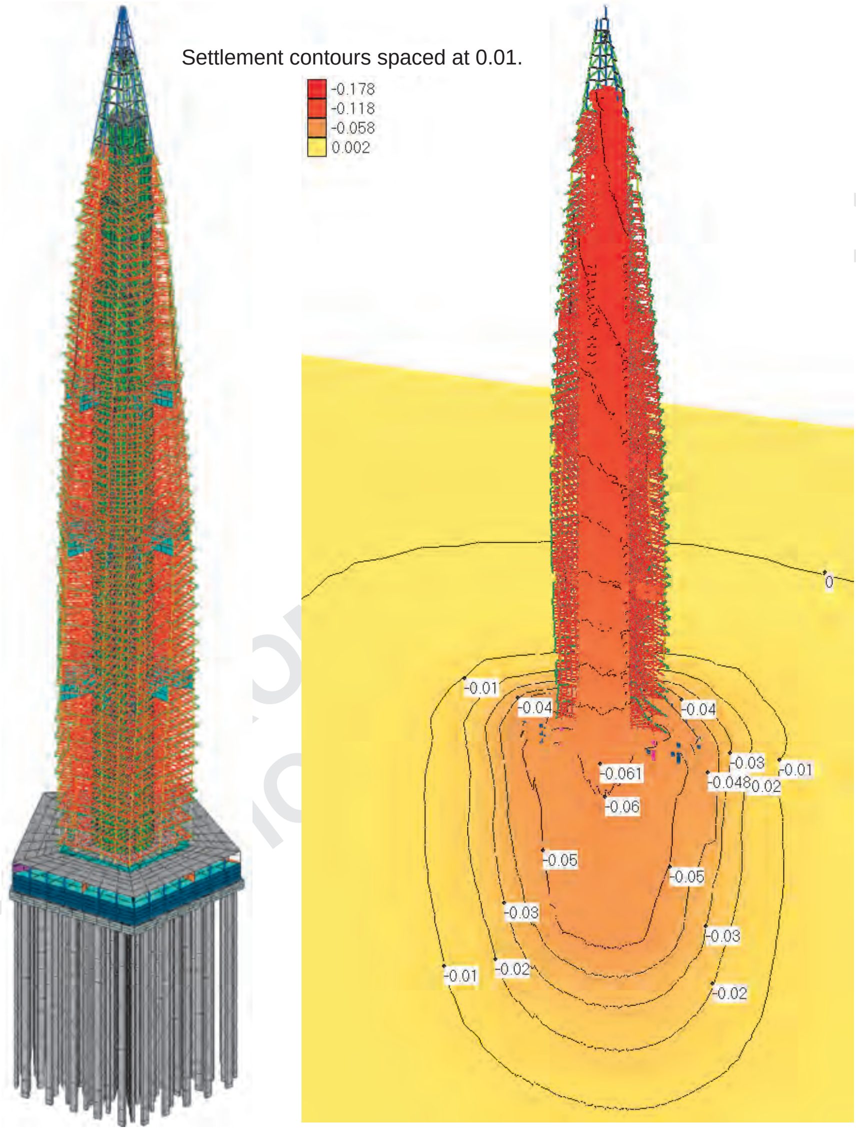
So why is it that over the 300-year long history of our city there appeared no high-rise buildings? The answer is simple: the soils are difficult. For high-rise construction in St. Petersburg to come to life, first of all, it was necessary to overcome the illusion that a 30-storeyed building is the same as three 10-storied buildings standing on each other's shoulders.

An increase in the floor number brings about not only quantitative, but also qualitative changes to the principles of design. Any skyscraper should have a uniform single core of rigidity, rather impressive columns on the periphery and the trusses occupying the whole floor, which connect the core with the columns at every ten floors of the building's height. That is why all skyscrapers are so similar to each other. The architect has practically no room for manoeuvre. The basic solutions are dictated by gravity, which is accompanied by an obedient designer and an equally obedient geotechnical engineer.

Certainly, not all architects are prepared to agree with such subordinated position. One high-rise construction architectural competition featured several ideas completely unfeasible in the light of the laws of gravity; there was even one flat skyscraper, theoretically capable of generating such wind turbulences that not only people, but also motorcars would have been sent flying through the air around it, were it ever to be constructed. Only one contestant came up with a physically realistic solution, and by that alone became the winner of the competition.

And preservation of St. Petersburg skyline is not something which we would like to discuss here. Let the experts decide. We shall talk instead about soils. Because a skyscraper remains a skyscraper even if it were to be constructed in Africa. All specific features of its design and construction are dictated by the local ground conditions.

In St. Petersburg, except for its most southerly suburbs, it is impossible to reach the bedrock with piles. One has to "rely" on what is available. And the best of what is available are the Proterozoic deposits. But even those deposits are not always capable of sustaining skyscraper loads, unless they are "conned" into doing so. If under a skyscraper one were to arrange a developed underground scope, the load on the subsoil could be reduced on account of the weight of the excavated ground. Thus, construction of skyscrapers in St. Petersburg soils directly depends on an opportunity to build a well-developed underground scope underneath. However, that underground space would not be so convenient, because it would be necessary to construct multiple rigid walls, standing in the way of cars, equipment and machinery. It was that solution that we had proposed for the skyscraper to be constructed on the banks of the Neva River. We conducted unique, unmatched soil-structure interaction calculations for the high-rise building in question. The calculation scheme for the building which has no identical floors was assembled in the most scrupulous way. We meticulously modelled the piled foundation consisting of heavy-duty rectangular barrettes, simulated nonlinear behaviour of subsoil, and computed development of deformations in time.



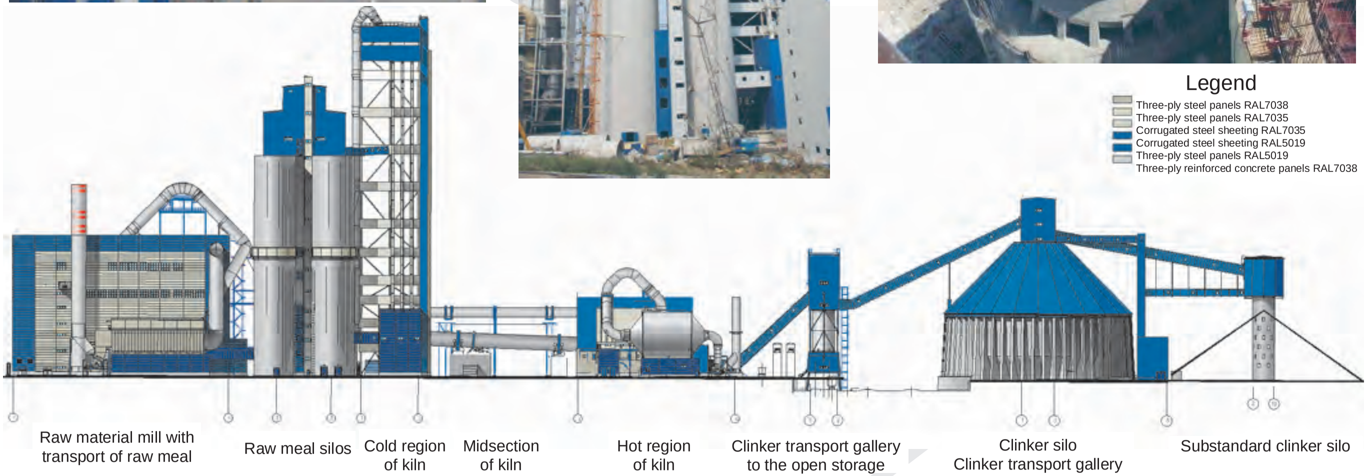
The settlement scheme of a high-rise building and settlement contours (m) according to calculations in view of nonlinear subsoil behaviour and soil-structure interaction.

Calculations demonstrate unequivocally that it is possible to construct such a building in St. Petersburg. Even if the foundations are going to be a little on the expensive side.

It is, however, necessary to warn investors that any skyscraper on any soils is unprofitable by any account. Especially, on soft clays. Appearance of skyscrapers always served to express ambitions of their owners. Today, when skyscraper technology all over the world has become ordinary and stereotype, such expression of ambition has become, shall we say, not awfully original. Today, something like the Summer Gardens with Roman sculptures and a modest, but graceful Summer Palace would be much more ambitious than a skyscraper. And we can console the opponents of high-rise construction in St. Petersburg with the following statistics. Over the last 5 years we have designed about 15 buildings with height of more than 100 m. So far not a single one of them has been even started.

The Cement Plant

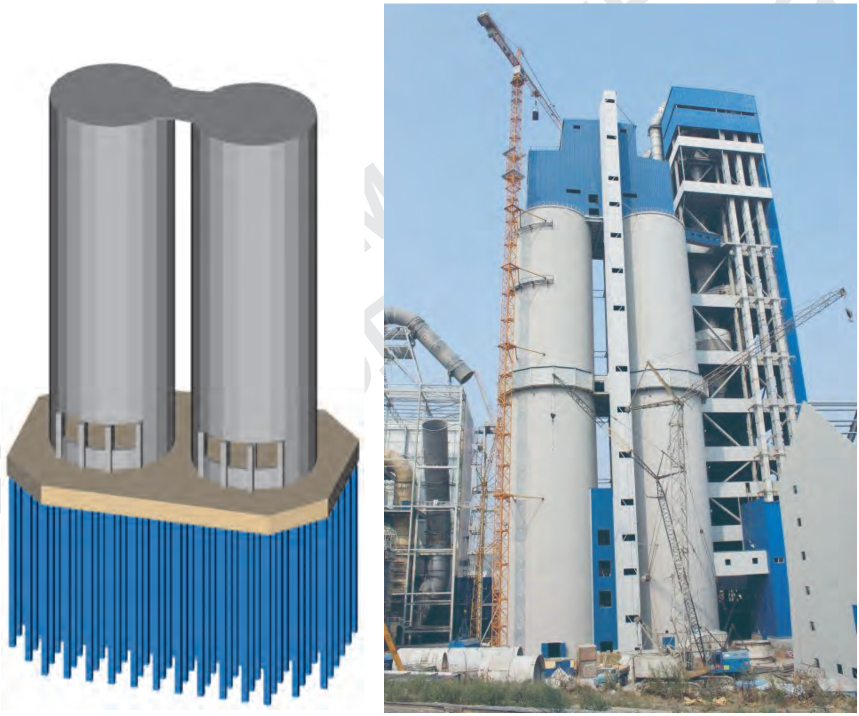
The reader who has had enough patience to have gone so far in our book might have had an impression that the authors and their colleagues from "Georeconstruction" Institute specialize exclusively in civil engineering and urban construction. This is not so. Industrial construction is an equally exciting field of expertise of the majority of our highly experienced designers. Our colleagues have designed a great many well-known industrial projects in Russia. For example *Cherepovets Smelter Plant* (almost entirely), *Bratsk Pulp and Paper Mill, etc.* In 1990s they designed *Baltica* and *Vena* breweries (apart from breweries, there was no other industrial development ongoing at that time).



Industrial architecture –a Cement Plant.

The building boom being at an end and during the times of recession our knowledge in the field of industrial architectural and structural design was again in demand. Recently we have designed several oil refineries, five rather large cement plants and prepared reconstruction design for *Bratsk Pulp and Paper Mill*.

There is a certain kind of charm about industrial design: there is nothing more complicated than designing industrial enterprises. In this field there are almost no competitors left. And also: development of Russian industry instils some hope for our country to become not only a raw material but also an industrial power. It feels good to be a part of the process.



The calculation scheme of silo structures and the constructed "industrial skyscrapers": 130-meter tall cold end of kiln and 80 m tall silos.

A cement plant can push one's imagination beyond the known borders with formidability and enormity of technological solutions.

Here is a 130 m high skyscraper. Raw material is fed to its top, undergoes necessary preparation inside and then fed into the kiln. This industrial skyscraper has an interesting name "cold end of kiln". Gravitational loads inside are not as large as wind loads.

After sintering in the kiln clinker is transported to the storage facility. It is shaped like a "cup" of 75 m diameter and 25 m height. Clinker is poured inside in a mound up to 50 m high. Pressure from such "cup" rendered onto the subsoil reaches 80 t/m^2, like from a serious skyscraper.

Clinker storage facility: a dome 100 m in diameter.

From the storage clinker is transported to mills, ground, mixed with various additives and then comes into silos, which look like twin-towers 80 m high and 22 m in diameter. Railway and motor transport arrives under the towers for loading. The towers are unloaded pneumatically. There are special clinker-feeding channels. Because of this the ring wall of the silo has to assume not only tensile but also bending loads. Such structure can only be constructed of prestressed concrete. The silo is like a "jumping" skyscraper: it is either full, or empty all the time. To design a foundation for such variable loading pattern is no easy matter.

Clinker storage facility: the inside view.

The plant's technology is state-of-the-art. It represents the so-called dry way of cement production. The plant is built as a single technological line, allowing to produce 6 thousand tons of clinker per day or more than 2 million tons of cement per year. The older technology – the so-called wet method – has three times less efficiency. The dry way allows to save energy, and consequently the cost of cement is lower.

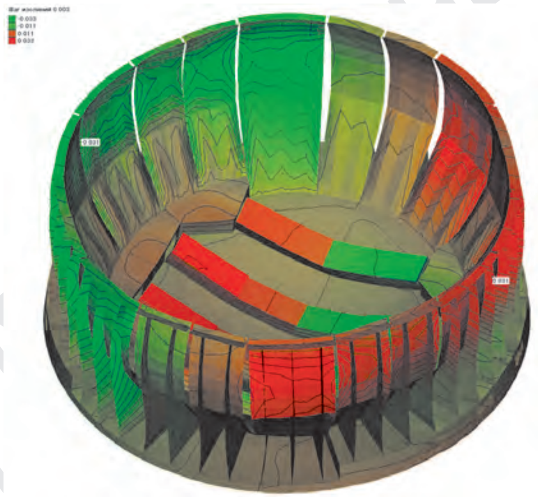
One such plant is being constructed today in Mordovia, other three – near Novorossiysk, and one – near Volgograd.

In Mordovia we encountered a rather exotic soil. A thick stratum right from the surface is composed of loams with very low density (16 t/m^3), fully saturated, but possessing rather high degree of hardness. Our native St. Petersburg loam turns into a jelly already at density of 19 t/m^3. What then is the secret of those soils? Are brittle destruction pattern and loss of structural stability relevant for them? Is formation of settlement troughs around buildings possible on these soils? As always in such situations, Professor Regina Dashko from St. Petersburg Mining University comes to our aid. There are no soils on the planet, no geological phenomena which her consulting power could be daunted with.

It is necessary to say that for a long time already we have stopped trusting site investigation performed by others. The responsibility of the designer is very high. Better safe than sorry, you know… Site investigation on all our projects is conducted only according to specifications developed by us, because we, geotechnical engineers, know, what kind of soil parameters will be necessary for our calculations. And we always hire a competent specialist (Regina Dashko's disciples be thanked!) to keep an eye on our site investigation.

Well, it turned out the mysterious soil was a kind of silica clay, formed by mineral particles of very low density. These soils do not possess settlement properties, but are still incapable of assuming significant loads. These are underlain with black clay, which owes its blackness to presence of natural bitumen. It looks firm, but owing to the bitumen "greasing" shears too easily (its shear strain modulus is low). Well, how about building something on such soils, then?!

For the heaviest structure – the clinker storage facility – we designed 45-m long piles having their toe levels in black clay. The design of the "cup" was done in such a smart way that it allowed the "cup" to undergo practically any settlements without any danger of damage. But the settlements were curbed with the adjoining technological galleries, as excessive settlement could disrupt the technological process.



The "flower" on the calculation scheme is the clinker storage facility 75 m wide and 25 m tall. It is filled with clinker whose "heap" reaches 50 m height. The figure presents the deformed scheme (the scale of deformations is greatly enlarged).

The 130 m tall skyscraper (the cold end of kiln, remember?) was designed on a ribbed slab with rib height of 6 m (and thickness of the slab between the ribs of 1,5 m). Very close nearby there were the 80-meter tall silos (the "jumping" skyscraper). It is not every day that you need to solve a problem of how it is possible to rule out tilt of a lightweight 130-m tall skyscraper towards a heavyweight 80-m tall one. The silos were founded on piles 45 m long and 1,2 m thick.

But all complexities of designing a plant in Mordovia tarnished before those of Novorossiysk. It would seem the subsoil here is half-rock, the so-called marl. But the site of the plant is hilly land, with differences in heights between neighbouring terraces of up to 20 m!

A 20 m tall retaining wall; a terrace of cement production plant in Novorossiysk.

To this seismic activity force 8 had to be added, making one liable to arrange the special anti-seismic joints dividing complex structures into simple blocks. Add also strong winds three times exceeding the force of our familiar Baltic winds. But the most unpleasant reality is that the half-rock marls are broken into blocks by four systems of fissures. Cracks can be of infinitesimal width or slightly more, and are filled by clay greasing. Durability of a separate block in this context is immaterial. The entire system should be considered as macrofragmental soil capable of slipping along the fissures. Injecting it is useless on account of clay greasing. So on the 30 m high sliding slope we had to build the clinker "cup" of even greater height. To ensure stability of slopes and structures constructed on them in conditions of seismic activity it was necessary to design a multiple system of retaining walls of several types – from a simple gravity wall to a complex one constructed tier-by-tier from top to bottom and strengthened with "Titan" anchors.

After such work a civil building starts to look like something very uncomplicated. This example was again demonstrative of how fruitful our approach to soil-structure interaction calculations actually was, allowing to unite efforts of the geologist, the geotechnical engineer and the designer with greater efficiency.

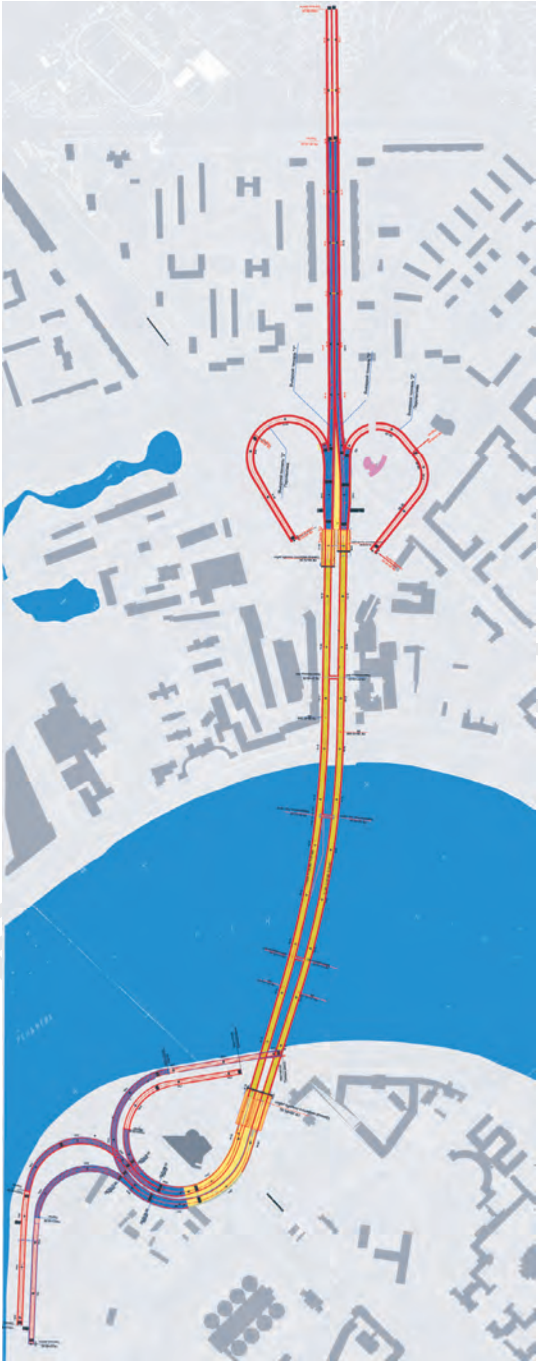
The Orloff Tunnel

The Orloff Tunnel was included into the general plan of our city in the 1950s. On the right side of the Neva River the intention was to arrange wide streets, on the left – vacant territories. The first steps towards the construction of the tunnel were made in 2006 when engineering designers of PSO "GALS System" and Moscow Metrogiprotrance developed the original concept. On the right bank of the river they planned to arrange the starting shaft in which to immerse the Tunnel Boring Machine (TBM). It was planned that under bentonite protection (see Part Three) the TBM should make the first tunnel, re-emerge on the left bank, turn back and commence travelling in the opposite direction, creating the second tunnel. "Georeconstruction" Institute was entrusted to design the turning chamber and the ramp (the "entry-exit" section of the tunnel) on the left side of the river. The turning chamber for the TBM for technological reasons has dimensions of 67 m by 50 m and the depth of 30 m. It is possible to immerse a full 10-storied apartment house in such a pit and you would not even be able to see the roof. Not only had such ambitious excavations never been constructed in St. Petersburg, they had never even been conceived of.

Fortunately, the geology of the site appeared to be absolutely uncharacteristic for St. Petersburg conditions: the upper layers down to 20-30 m from the surface were compounded entirely of sands. It was not for nothing that this area used to be called "The Sands" in the olden days. In such geological conditions the modern western geotechnologies – such as the diaphragm wall and jet grouting had been well tested. But there was the other side of the coin, inasmuch as sand is highly permeable. Excavating a pit in sands, right on the riverbank could only be possible in a perfectly water-tight cofferdam. To limit water-intake into the pit two solutions might have been used. The first solution would have been to construct vertical screen cutting though the layers of sand and reaching impermeable clays. The second – to provide a layer of jet-grouted soil at some depth to block off the ground water. In the latter case the vertical elements of the pit could end slightly below the edge of the grouted layer. However, in that case the building contractor would have been held hostage to the quality of the resulting product. A presence of any "hole" in it would have made questionable the very possibility of excavation. The second problem would have been the pressure of water from below, as the grouted layer does not have enough strength to withstand bending deformations. Therefore, a layer of grouted soil no less than 2 m thick would have been necessary, loaded on top with 6…7 m of unexcavated material, to counteract buoyancy. A simple calculation of required depths eliminated our hope to reduce the length of a vertical section of the cofferdam. So, the second option appeared less reliable and more expensive in comparison with the first.

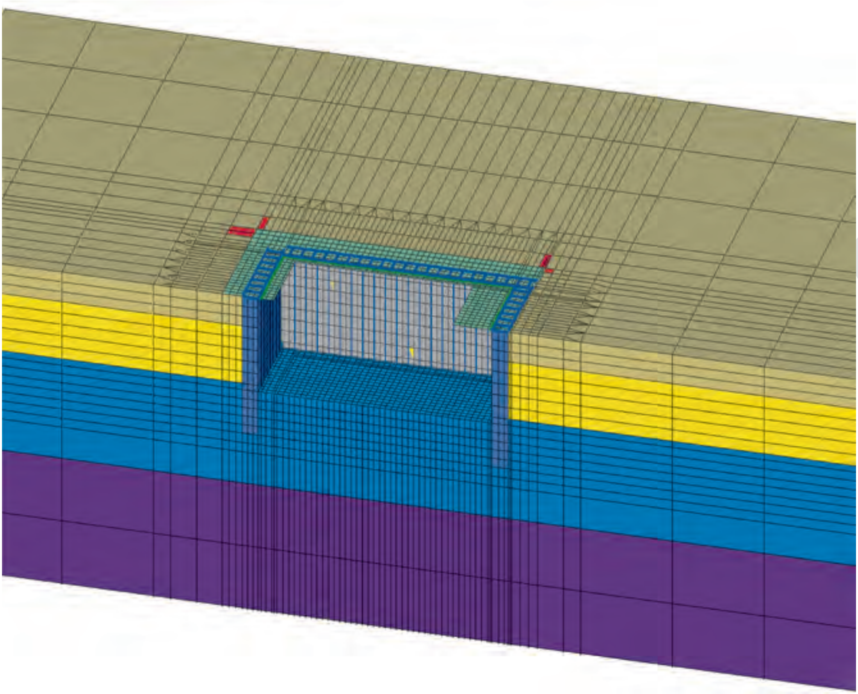
Well, it was necessary to arrange a cofferdam down to the level of impermeable clays. Sheet piles could not be driven to the necessary 40-45 m. A secant or contiguous pile walls usually leak like a sieve. Moreover, the chinks between the piles in such a wall can form also below the bottom of the pit where it would have been impossible to find and seal them. Therefore there was no other choice apart from the diaphragm wall.

But how to provide stability of a 30 m deep cofferdam?



The Orloff Tunnel layout.

Arranging struts here was very unreasonable. They needed to be put in with the pace of 5 by 5 m, with piled supports. It would obviously have been impossible to turn the TBM in such conditions. It would have been also necessary to provide additional lining to assume horizontal soil pressure. Unnecessary, lengthy and labour-consuming work… Anchorage, perhaps, instead of struts? In sandy soils anchors can be rather effective... But they surely would have blocked the exit and re-entry of the TBM. Therefore, there was the only one way out: the turning chamber cofferdam with one level of shoring on top. This could have been constructed of a powerful waling beam 15 m wide and 3 m high. The cofferdam wall had to go down to 44 m. Exactly in the middle of the walls enormous loads of 4000 t/m would have appeared. The only way to assume such loads was to construct the diaphragm wall of H-section. A diaphragm wall of such section was not at all easy to construct because at the slightest infringement of technology the ground in the corners of the H-section would pour inside the trench. But such layout of the cofferdam possesses extremely high rigidity which makes it rather optimistic for St. Petersburg. Therefore the companies "Geoizol" and "Franki" (Belgium), under scientific supervision of the present writers, made several trials of constructing diaphragm walls of complex cross-sections on several test sites in St. Petersburg (we spoke about it in Chapter 13), including two sites on the left-bank ramp section of the Orloff Tunnel.



Calculation scheme for the cofferdam with plan dimensions of 67 by 50 m and depth of 30 m (cross-section). The cofferdam is constructed of H-section diaphragm wall, creating a very rigid 6.6 m thick box-shaped structure. A powerful waling beam on top is used as the strutting option.

While designing the cofferdam for the TBM turning chamber, it was very important to define time development of its horizontal displacement. The only computing tool capable of solving such problem is the viscoplastic model incorporated in the library of our *FEM models* software package. According to the calculations performed in spatial setting, over one year after excavation the horizontal displacement of the cofferdam can reach 10 cm. If works extended in time, the horizontal displacement would keep increasing. Therefore it was necessary to limit the works timeframe to one year.

The first ever successful trial of diaphragm wall protected excavation in St. Petersburg on the Orloff Tunnel site.

The design for the left bank section of the Orloff Tunnel project developed by "Georeconstruction" Institute was regarded rather highly by the State Expert Board of Russia. Two trial sections were successfully constructed. We had solved all technical problems. Likewise, PSO "GALS System" and Moscow Metrogiprotrance had resolved all technical issues related to the actual tunnelling, and Lenmetrogiprotrans had done everything for the right bank section. The only thing remaining to be done now is to construct the tunnel itself.

Afterword

Dear Reader, we have tried to tell you about Geotechnical Engineering in a clear and intelligible manner, avoiding scientific complexities on the one hand, and popular oversimplifications on the other. We have shared with you our long-term experience and have attempted to give you some advice, aspiring not to sound homiletic or didactic. We hope that this book will help you to get properly oriented in the field of Geotechnical Engineering.

Some people think that soil is an unknowable matter. We do not think so. Such is the opinion of the ignorant. Geotechnical Engineering is an exact science. It is important only that it is wheat that finds its way into geotechnical millstones, not chaff. And this our work is your Guidebook in the world of Geotechnical Engineering.

Dear Reader, if you are a creative person, if you aspire for your buildings to stand fast for hundreds of years, you are advised to read and re-read this book.

We organize and host many international conferences and workshops on Geotechnical Engineering, where you can come to know the achievements of the best specialists in the field today. Such knowledge allows us to be on the cutting edge of scientific and technical progress. We invite you to our conferences and workshops. We invite you to mutually fruitful cooperation.

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V.M. Ulitsky
A.G. Shashkin
K.G. Shashkin

The Geotechnical Guidebook

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and underground structures)

Second enlarged edition

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