Saimaa University of Applied Sciences

Technology, Lappeenranta

Civil and Construction Engineering

Elena Bogdanova

Soil improvement by the method of microblasting

Bachelor's Thesis 2012

Abstract

Elena Bogdanova

Soil improvement by the method of microblasting, 58 pages, 5 appendices

Saimaa University of Applied Sciences, Lappeenranta

Double Degree in Civil and Construction Engineering

Bachelor´s Thesis 2012

Tutor: Sami Laakso, Saimaa University of Applied Sciences

The purpose of this project was to study a new method of soil improvement by microblasting and the identification of the advantages and disadvantages of this method.

The information was gathered from literature, newspapers, articles, the Internet and by interviewing one of the authors of this technology- Rishard Imiolek. The first part of the study was carried out in February and April 2012 at Polbud-Pomorze Company (Poland) which is the owner of the patent for this technology,. Further study and familiarization with the process of soil compaction designing by using this method were conducted in an office in Russia in March and May 2012.

The results of the study show that the method of microblasting is one of the most effective and innovative methods of compaction of weak soils. Based on the theoretical study and the example of using this method in real life the disadvantages and advantages of the microblasting method have been identified.

Keywords: explosive charges, compaction, ground improvement, subgrade strengthening, consolidation, borehole, settlement.

CONTENTS

1. Introduction

The urgency of the problem of road construction in complex conditions in areas with specific types of soils (peat and organic-sediment, sludge of different origin, clay, salt marshes, wet, etc.) is still relevant at present time.Excessive moisture, low bearing capacity, high compressibility, and other negative qualities of these soils make process of road construction more expensive and difficult.Such poor soils are formed at the crossings of the floodplain of rivers, estuaries, lakes, oxbow lakes, etc.Road construction in the areas of occurrence of weak soils is a complex multidimensional problem, which includes study of the properties of soils and conditions of their formation, taking into account the specifics of the road construction. One of its features is technology research.

The building methods used in earlier times in most cases focused on the removal of soil from the base of the weak subgrade with improved road surfaces. In areas of considerable length the excavation workload, complexity and expenses become extremely high. A significant rise in the latter and, last but not least, the low rate of construction stimulated the research of new methods of soil reinforcement.

Nowadays new researches are carried out. They are based on the hypothesis of the possibility and feasibility strengthening of the soft ground at the base of the subgrade, provided that the design and technological solutions will be made on the basis of specific properties and conditions of occurrence of these soils.

In the first part of the thesis the most common ways of weak soil reinforcement will be briefly observed. Such methods as dynamic replacement, jet grouting, sand piles and microblasting technology will be marked in this paper.

The attention will be focused on the microblasting method as the most innovative one. Design and realization of soil reinforcement using microblasting technology will be reviewed in the second part.

The final part will include comparison of two ways of soil strengthening based on a real project. As a result of this discussion the most effective one will be found and the advantages and disadvantages of this method will be determined.

This paper, which is based on the study of various ways of soil strengthening and their comparison, will attempt to determine the most effective method.

2. Ways of weak soil reinforcement

2.1. Dynamic replacement

Dynamic compaction is a technology of subgrade strengthening consisting of non-cohesive sand soils. The essence of technology lies in the fact that a weight of 5-20 tons is lifted by crane to a height of 10-30 meters and reset to the amplifying ground. Compaction is performed several times at each point of the subgrade. Craters, which were carried out as a result of this process, are backfilled with a mixture of sand and gravel. (Fig. 1) The effectiveness of dynamic compaction depends on soil conditions, as well as the weight of the cargo and height of the cargo drop. The effectiveness increases with the increase of cargo's weight and drop height. This technology is used to strengthen weak soils to the depth of 14 m (in practice up to 10 m high).

Figure 1 "Crater" which was formed as a result of the cargo drop from the crane

A variety of dynamic soil compaction is Dynamic Replacement. The stone columns are performed in the case when weak soils are situated not very deep in the ground (maximum of 4-5 m from the surface). The same equipment as in the dynamic compaction is used in the method of Dynamic Replacement. The cargo, hitting the ground, makes the crater, which is then filled with coarse-grained material. Aggregate material may include sand, gravel, quarry-run rock, concrete debris or blast furnace slag. As a result a large diameter column of 1,4 to 2,5 m is created. This process is repeated several times and as a result stone columns are formed.

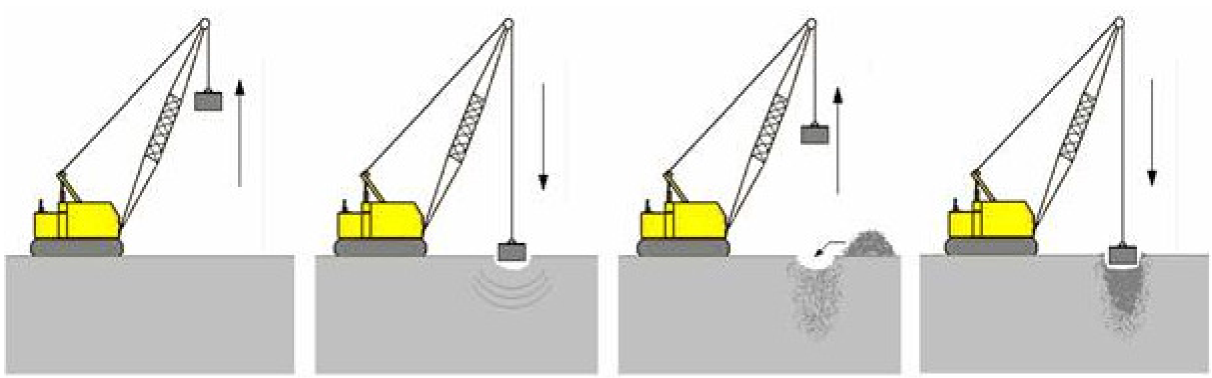


Figure 2. Stages of the Dynamic Replacement process

Polbud-Pomorze also employs surface dynamic compaction technology. This improvement technique utilizes repeated applications of the impact delivered by a special, flat tamper, weighing 12 tons, free falling from heights of 5-10 meters. They use unique, special barrel-like shape of the tamper which enables lateral expansion of the column, which accelerates consolidation and improves mechanical characteristics of soil between the columns. Such a tamper is presented in Figure 3. A single or double tamper releaser is mounted on the upper part of the tamper.

Figure 3. Special barrel-like shape of the tamper

In some cases, the location of the Dynamic Replacement Columns can be pre-excavated. The excavation is then partially backfilled with a “plug” of granular

material. This initial plug is then driven into the soft layer by the tamping energy of the weight to the required depth of treatment. This crater is regularly backfilled with additional granular material between each tamping phase.
Pre-excavation is used in the following cases:

- Presence of dense and compact layers at the surface;
- Need to reach deeper soft layer and create longer Dynamic Replacement Columns;
- To minimize heaving of the surrounding soils.

The backfill material can be either placed over the whole treatment site before pounding, thus creating a working and load transfer platform once the work is completed, or stockpiled at regularly spaced intervals across the site.

Advantages:

- Dynamic Replacement is well adapted to substantial loading conditions (up to 150 tons per column) as well as under embankments to improve the factor of safety against slope failure. With this technique, replacement ratios of up to 20-25% can be achieved;
- Dynamic Replacement Columns can be used in peat or in soils with high organic content without the risk of bulging due to their relatively low slenderness (ratio of height over diameter);
- Dynamic Replacement Columns can increase the time rate of consolidation due to their draining potential;
- Very efficient and high production rate can be achieved.

2.2 Jet grouting

This technology is extensively used for soil improvement and in construction works such as founding of industrial and large-size buildings, forming sealing slabs, underpinning, pile walls as well as securing of slopes and scarps. It can be applied universally in any type of soil and is perfect for urban areas, buildings and under railway and road structures.

This technology consists of mixing soil with stabilizing slurry (bonding grout), injected under a high pressure. A wall or cylindrical column is formed after bonding the grout mixed with soil. (Figure 4)

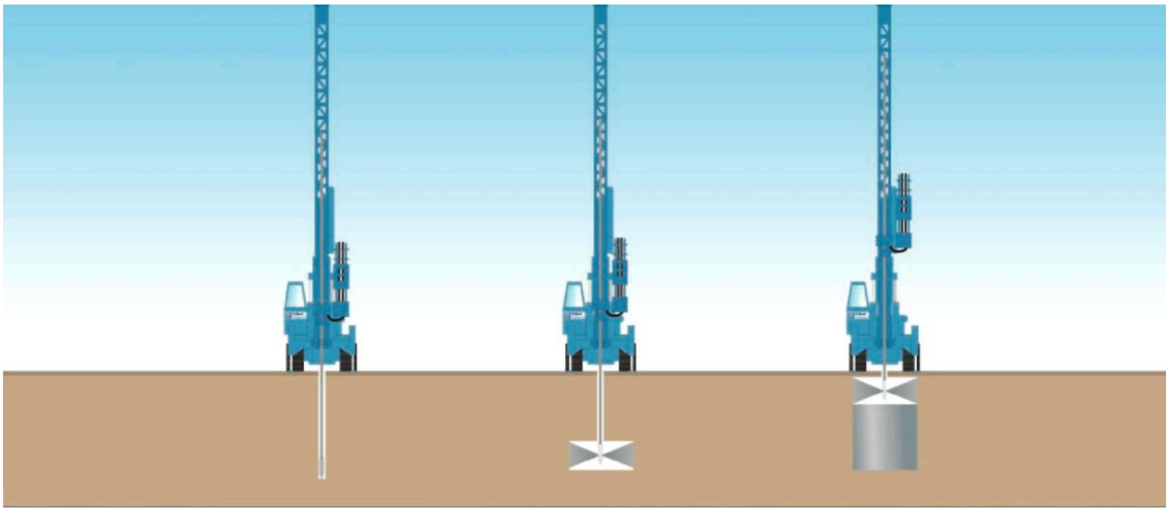


Figure 4. The process of jet-grouting

The process starts with drilling a hole with a drilling rod equipped with jetting nozzles. When the drilling pipe reaches the required depth it is pulled back while rotating and injecting the bonding grout supplied under a high pressure. This bonding grout and soil mixture form a highly resistant and impermeable structure.

They can be reinforced shortly after forming the structure. Jet grouting elements may be constructed at any angle and form various configurations e.g. columns, palisades or slabs.

A feature of this method is rather complex equipment.

Destruction and mixing process require a high value of the kinetic energy of the jet of the slurry. So for realization of jet grouting process it is necessary to use a powerful high-pressure pump. Experience has shown that the pressure of injection should be between 400 to 700 atm.

Another important part of the equipment for grouting the soil is a monitor, equipped with nozzles. Due to the high abrasive properties of slurry nozzles are made of special metal-ceramic composition.

The strengthening of soils by jet- grouting method is used in the following cases:

- if there are weak water-saturated and peat soils at the base of structure, which have modulus of deformation E ≤ 5,0 MPa;
- in the case of large loads on the foundations (0,3 MPa and more);

- In the case of contiguity to the existing building and construction of new buildings.

Advantages

- Jet grouting offers a fast rate of work implementation
- Jet grouting elements may be constructed vertically, diagonally and horizontally in the ground, in a variety of available diameters and different length
- It enables piling or subsoil improvement in the grounds where obstacles such as old foundation, rubble, boulder and rock debris are present
- It is the only technology – besides micropiling – which enables stabilization of already existing foundations and grounds made of reinforced concrete slabs (e.g. airfields, embankments etc.) without the need of dismantling
- The soil-cement elements have a high compression resistance
- Structure homogeneity and lasting chemical stability of the jet grouting structure are ensured
- This technology does not cause soil degradation.

Also jet- grouting is used for the construction of Grout curtains.

Construction of foundation pits in stable soils is not a difficult task, but it becomes more complex at times when a soil mass is watered. In such cases a traditional method is used: a method of constructing a barrier in the soil or a sinking caisson stretching down to the depth of natural confining layer occurrence followed by excavation of soil under protection of slurry walls.

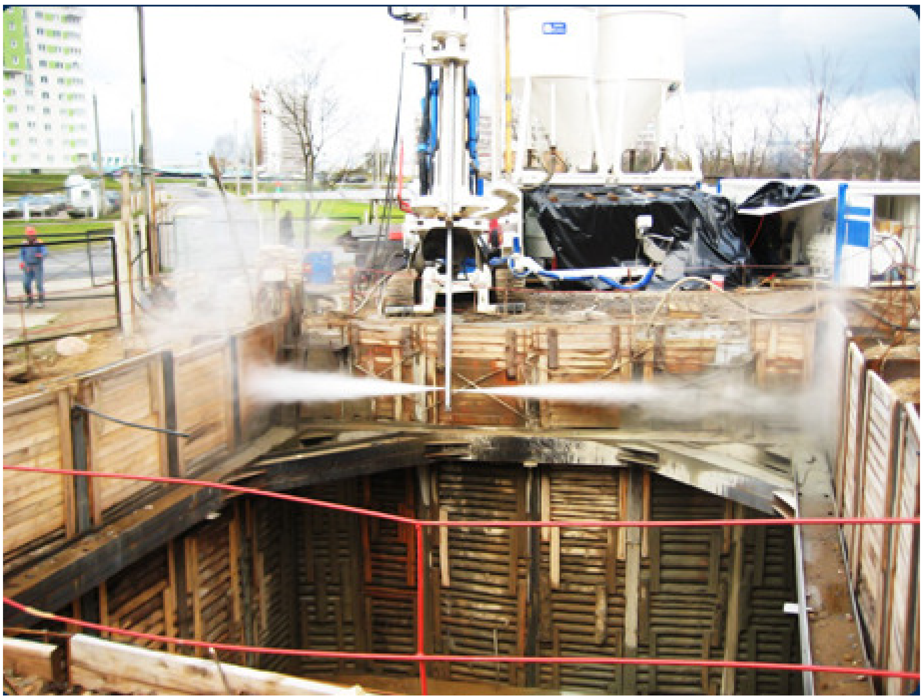


Figure 5. Jet-grouting technology in action

Sometimes there is no natural confining layer or it is so deep down in the ground that reaching for it with enclosing structures is just economically impractical. Until recently methods of dewatering were used in cases like these. Such methods were often ineffective when dealing with the soils of high filtration properties or became unsafe due to underwashing that took negative toll on foundations of near-by structures and buildings.

Today, this problem can be solved successfully with the help of the jet grouting technology, that allows to create an artificial confining layer, also known as horizontal grout curtain (HGC).

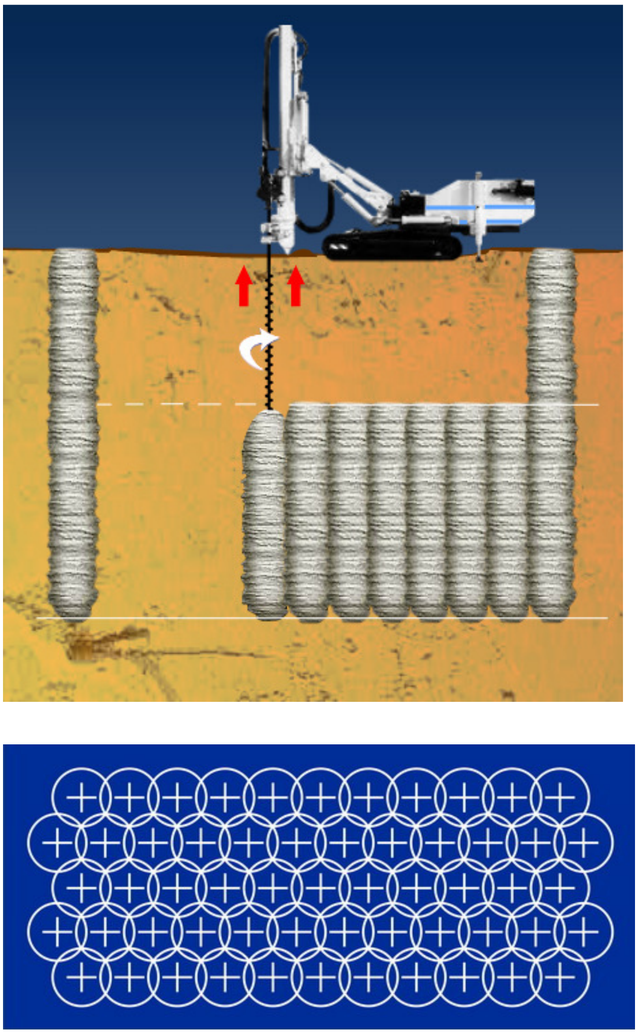


Figure 6. Construction diagram of a horizontal grout curtain

Construction of curtains is performed from daylight surface level of a construction site or from a level of intermediate mark at a pilot pit. The depth of a pilot pit is limited by the groundwater level. The latter option not only decreases the metreage drilled, but also makes it easier due to removal of surface soil, which may contain remnants of previous foundations, construction debris, etc.

This method helps improve deformative properties of the soil and furthermore homogenize those properties substantially within a given building footprint. At the same time it reduces the chance of relative settlement, which in turn decreases chances of defects and damage.

2.3 Sand pile drains

Pile drains are constructed in order to assist squeezing pore water out of compressed layer of waterlogged soft ground. Pile drains are wells, filled with sand or any other filtering material. Compacted sand columns are analogous to pile drains and are used to reduce settlement and elastic vibrations of the soft ground.

Pile drains and compacted sand columns are placed within triangular, square or staggered grid and spaced 2-1 m for drains and 1-2 m for columns. Sand with coefficient of filtration not less than 6 m/d or sandy gravel with grain size up to d60 mm are used to fill up drains. The lower part of the platform (working platform) is made of drainage soil with coefficient of filtration more than 3 m/d.

Compacted sand columns are made of sandy soils that are suitable for stacking and have no additional restrictions. Filling requirements for compacted sand piles that will be used as drains are the same as those for pile drains. Drying and sealing properties of drains and columns increase if enhanced with lime filler.

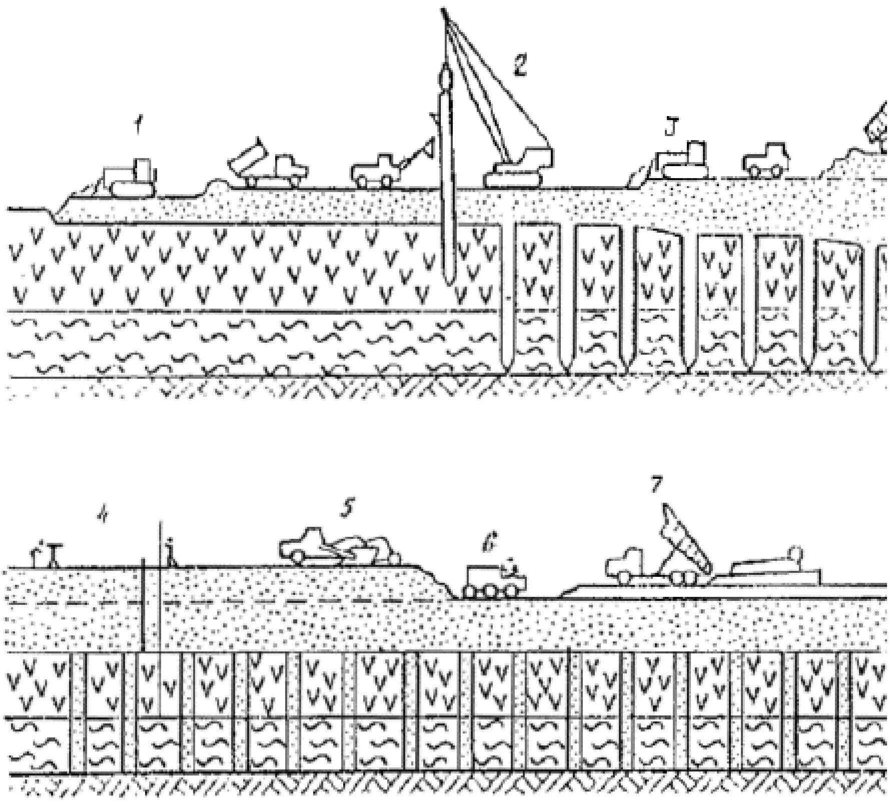


Figure 7. Process diagram of constructing a passage with pile drains

The process diagram of constructing a passage through soft ground deposition with pile drains can be seen in Figure 7:

1 - shifting of the working layer with a bulldozer; 2 - construction of drains and filling drains with sand; 3 - ramping the earth bed up to the formation level; 4 - checking the settlement; 5 - removing of the subcharging layer; 6 - postcompaction of the earth bed; 7 - construction of the road pavement.

the diameter of sand pile drains and compacted sand piles depends on the equipment and length - 300-600 mm.

Before starting construction sand pile drains and compacted sand columns on the surface of a marsh, dumping of a working platform from sand is performed. The thickness of the platform depends on the bearing capability of the platform ground and weight of mechanisms, and is around 0.5 - 1 m. The working platform should be wider than the pile field not less than 2.5 m. Construction of the working platform should be performed according to the process diagrams, approved for constructing platforms on marshes.

First, a plan of the working platform is mapped, and then locations of well centers are determined, extended and fixed along with axes of traverse rows. Filling sand for wells is transported to the mapped working platform by dump trucks.

The construction process of piles and drains includes the following steps: driving of a pipe casing, filling it up with sand, vibro-extraction of the casing and sealing sand inside a pile. Piles are installed in rows of 20-30 by a machine that moves back and forth like a shuttle, turns around and moves backwards after finishing a batch.

The pipe casing is driven into soft ground with the help of vibration, vibration-free squeezing of the soil, which gets thin if exposed to vibration, or with the help of the method, combining these two technologies. It is rational to use a separate machine of an anchor truck type to penetrate the working platform and interlayers. When the pipe casing has reached a given depth level, it is filled with sand with the help of a loader, equipped with a clamshell bucket.

The pipe casing is extracted while the vibrator is still working. During the first ten seconds of the procedure the speed of extraction should not exceed 0.4 m at the maximum vibration intensity level. If sand is flowing freely from the pipe, the extraction is continued at the speed of 0.2 m/sec while lowering vibration intensity levels. After extraction is finished, the pipe casing is moved to a new mapped point.

Construction of piles at the foundation of a bed helps:

a) reduce normal levels of soft soil stress by allocating some of the stress load through piles to the bedrock;

b) reduce the maximum levels of shearing stress in the soft soil, and thus improve working properties of Yoldian clay in the shearing foundation

c) shrink the amount of final settlement of Yoldian clay and overlying turfs, thus reducing the duration of intense settlement phase of the foundation;

d) increase the density and decrease the percentage of moisture in Yoldian clay in the foundation by placing sand bank into the soil, which subsequently promotes further increase in soil strength factor (soil adhesion and angle of internal friction);

e) increase the stability of the foundation in case of destabilization due to shearing along a fixed slickenside caused by partial replacement of clay at the slickenside with sandy soil that has high angle of internal friction;

f) accelerate settlement of turf, which is overlaying a clay layer, due to drainage of turf mass with sand columns, which function as pile drains if filled accordingly.

2.4 The microblasting technology

The microblasting technology is used for the improvement of subsoil under civil and hydro engineering structures. It employs high-energy explosions to modify the surrounding soil. The energy generated by the explosion of 1 kg of TNT is equal to the energy of 5 tons of tamper falling free from a height of 100 m.

Microblasting may be applied underwater – therefore it is perfect for the harbour areas, offshore wind farms, reclaimed islands, breakwaters etc.

It is also used for soil improvement under road embankments and airfield pavements. In saturated loose granular soils the application of microblasting technology guarantees a very effective and homogeneous densification which fulfills the safety requirements for important structures like nuclear power plants or dams.

Types of subsoil for improvement by microblasting:

- saturated loose granular soils (also offshore)
- soft fine-grained soils (peats, muds)

In the microblasting technology the energy is obtained from small explosive charges which are installed underground or on the ground. The mass of charges and their arrangement are carefully designed for the given ground conditions to meet the expected demands.

The types of Explosive compaction are presented in Figure 8, Figure 9 and Figure 10.

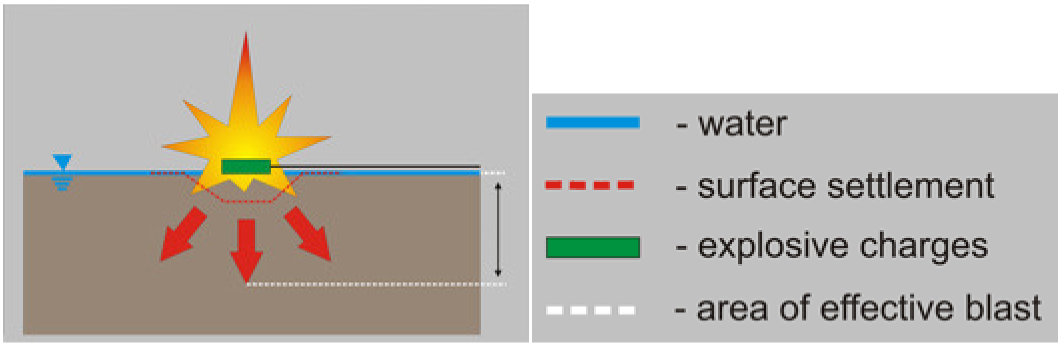


Figure 8. Explosive compaction with surface charges

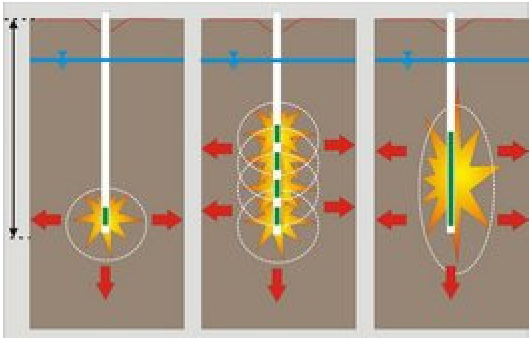


Figure 9. Explosive compaction with hidden charges

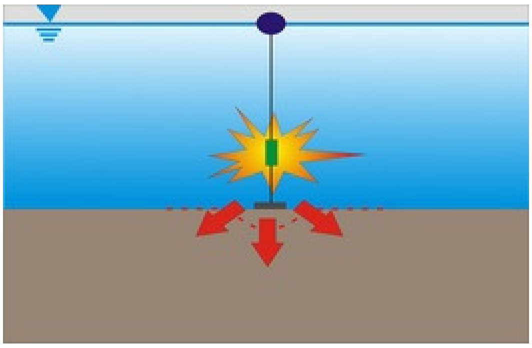


Figure 10. Underwater explosive compaction

In the case of explosive compaction of underwater soil deposits, the charges are installed closely over the bottom.

Details of this method will be discussed in Chapter 2.

3. Microblasting technology

3.1 How does it work?

The problems related to low stiffness and strength of dumped soils and the danger of liquefaction failures can be resolved by an effective compaction of the soil skeleton. As the dump bodies usually extend over large areas, the important question for economically and technically efficient ground improvement methods emerge.

The technology of this method is rather simple. Single or multiple explosive are placed in boreholes that are drilled to the necessary level of treatment. The soil compaction occurs quickly after the explosive charges are ignited.

A lot of successful practical cases with the application of ground improvement with explosive soil loading are known, where soil zones in a depth of h= (10…20) [m] have been compacted with a specific explosive consumption of $m_{e,sp}$= (10…35) [g/cm^3]. The typical distance between boreholes was chosen in the range Δr = (5…10) [m], which was influenced by the approximate horizontal extension of the compacted soil zone, by the size of the explosive charge, the initial degree of water saturation and the initial density of the soil skeleton (Gohl W.B., 659).

For loose, saturated, cohesionless soils, the detonation causes a rapid water pore pressure increase and destruction of the original soil fabric structure. After a relatively short time (about 3 weeks), pore water pressures dissipate, and a new ground mass with a higher density ratio is created.

For weak cohesive and organic soils, the improvement mechanism is different – microblasting creates sand columns made from a working platform of granular fill and other sand layers in the subsoil, which readily fill the caverns made by the explosions (Figure 11). The resulting columns are 0.6- 2.0 m in diameter, depending upon the applied mass of the charges. Blasting hole layout and charge detonation order are matched to ensure that water is forced out of previously formed, neighboring columns. Intensive water outflow from previously formed sand columns is easy to spot during ground improvement works and it is proof for effectiveness of Microblasting technology.

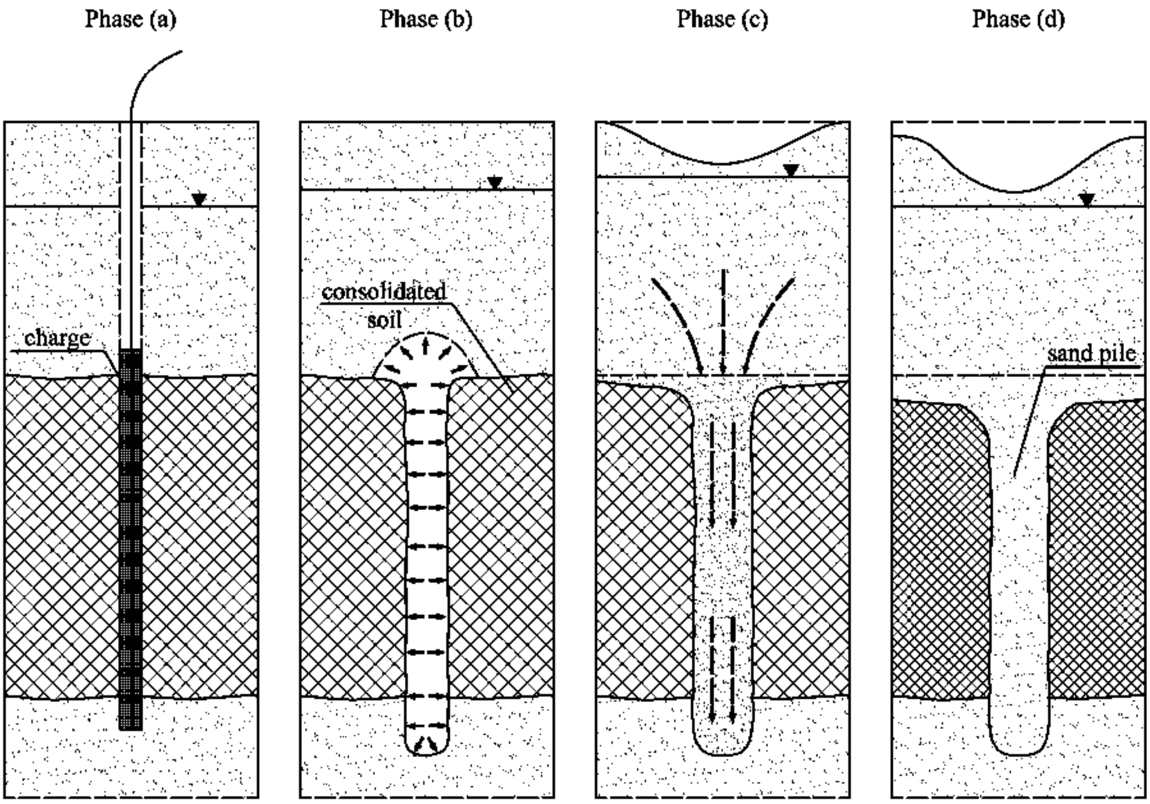


Figure 11. Scheme of weak soils' improvement using Microblasting technology:

(a) placement of blasting charge, (b) explosion and cavern's creation, (c) granular material's inflow into cavern,

(d) formed sand pile and crater shown on the ground surface

Compaction of cohesive and cohesionless fully saturated subsoil by explosions is one of the most effective methods of soil improvement, especially in large areas (e.g. tens of thousands of square metres) and depths to 30 m.

Immediately after the explosion the natural structure of fully saturated soil is destroyed and loosens. And only later does compaction of the soil gradually follow. This leads to an excess water outflow in the treated zone quick settlement of the ground and, in consequence an increase of the relative density index.

The following conditions guarantee the best compaction results:

(n) fully saturated subsoil

(b) content of gaseous phase should be less than 1 % of the total soil porosity

(c) mass and number of blasting charges should be selected in such a way that eruption and spreading out of soil mass from the boreholes is prevented.

The following factors influence the results of compaction caused by explosions:

(a) mass and shape of a single blasting charge

(b) number of blasting charges detonated at the same time

(c) the total number of blasting charges detonated in one series of explosions

(d) number of explosion series (most often, four to six series are used)

(e) localization of blasting charges in plan and vertical section of a subsoil

(f) property of blasting charges

(g) breaks between successive explosions.

Depending on the nature of the subsoils and on the thickness of the compacted layer, the duration of the consolidation process in cohesionless soils varies between 1 and 6 weeks after termination of explosion work. In the case of cohesive soils this can extend to 3 months.

3.2 Theoretical analysis

In most cases, the preparation of ground improvement measures is carried empirically, based on the results of test explosions and field tests noting the initial and consecutive state in the subsoil before and after the explosive soil loading respectively.

Existing theoretical models of explosive soil loading do not lead to results about the emerging spatial density distribution after the explosive impact,but they can significantly support the preparation of practical explosive ground improvement projects with the identification of the influence of soil physical and soil mechanical parameters at the improvement site and of explosive specific parameters on the effectiveness of energy transfer from the explosive source into the soil.

A practical theory has been developed by Tamaskovics. It allows the calculation of the mechanical properties in a quasi one-dimensional, radially symmetric strong shock wave-field induced by a single source of explosion with planar, cylindrical or spherical symmetry initiated in full-space of nearly saturated soil.

In the model, shock wave propagation is described in full space filled with a ternary mixture of a porous granular solid skeleton, containing a fluid and a gas phase in its pore space simultaneously. On the microscopic scale, the solid and fluid phases are considered as incompressible, the gas phase as compressible.

The granular skeleton is regarded to be compressible until full fluid saturation is reached.

Based on the theoretical results, the spatial soil zone influenced by the explosion can be determined. However, the calculation of resulting compaction effects to be expected in the granular skeleton due to explosive loading remains still an unsolved scientific problem.

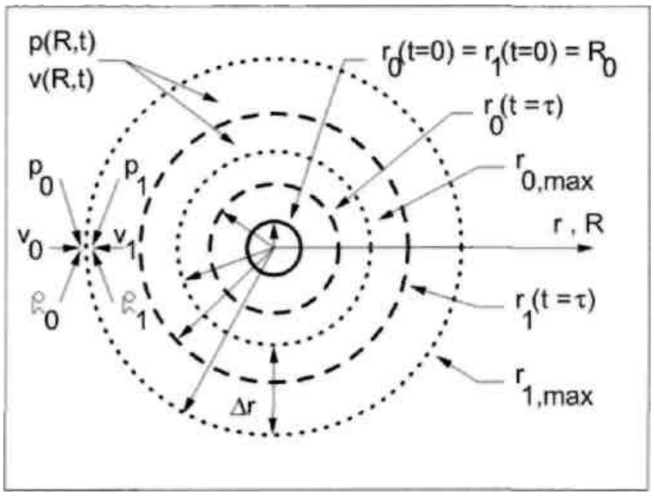


Figure 12. Theoretical model

In Figure 12, the theoretical model of shock wave propagation induced by the wall of a highly pressurized expanding cavity is demonstrated. At the moment of explosion, a strong shock wave-frontemerges from the surface of the explosive charge and propagates in full-space with the propagation velocity c(t). The spatial coordinate of the cavity wall is given by the function r0(t), the spatial coordinate of the shock wave-front by the function r0(t). Due to the sudden compression of the gaseous phase on the shock wave-front, the density of the soil is assumed to change from the initial density p0 to the final density p1, and to remain constant during further process of wave propagation. On the shock wave-front, velocity and pressure in the soil are also suddenly changed from their initial values of v0 and p0 to subsequent values v1, and p1 respectively. At a certain spatial coordinate r1max, the propagation of the shock wave-front is assumed to cease to exist. At this time, the expanding cavity of gaseous explosive products will reach the spatial coordinate r0,max

The analytical problem of shock wave propagation under the introduced simplifying assumptions leads to the ordinary differential equation

$$\ddot{r}_0^* + F^*(r_0^*, \beta, \nu)\ \dot{r}_0^{*2} = Q^*(r_0^*, p_0^*, \kappa^e, \beta, \nu)$$

that describes the dependence of the dimensionless spatial coordinate of the cavity wall of gaseous explosion products r0*(t*) on the dimensionless time t*. The dot placed over symbols denotes time derivative. The dimensionless influence functions F* and Q* in equation (1) depend besides of the dimensionless spatial coordinate of the explosive cavity wall r0*, on the initial volume fraction of the gas phase in the soil p, on the isentropic exponent of the gaseous explosive products Ke, the dimensionless initial pressure in the soil p0* and on the symmetry factor v, taking different forms for planar, cylindrical and spherical symmetry of the explosive charge.

The mathematical simplicity of the presented theory makes it suitable to be used as an initial condition for the calculation of the explosive induced wave-field in a nearly saturated soil. In order to formulate the initial condition, the particle acceleration, velocity, displacement and pressure fields are calculated for a given spatial coordinate of the shock wave-front, where it is assumed to cease to exist. It must be considered however, that the presented theory is restricted to deep underground explosions, as the boundary conditions from the free surface of the half-space are not taken into account.

The explosive compaction method can be used to increase the density in water-saturated granular or slightly cohesive soils (sands and silts) up to the intermediate density range in a very productive and economic way and is best appropriate for ground improvement objects extending over large surfaces and depths. At the application of the explosive ground improvement technique must be considered, that the surrounding environment can suffer a serious dynamic excitation, which can endanger the stability of existing buildings and earthworks. Furthermore, the applicability of explosive compaction is restricted to the water saturated soil region below the water-table level. Regarding the efficiency of ex-plosive energy introduction into soils, the presented theoretical model shows, that the soil zone influenced by the explosion is expanded by the increase of detonation pressure of the explosive, by the increase of the mass of the explosive charge, by the increase of the initial degree of saturation in the soil, by the decrease of the adiabatic exponent of gaseous explosion products and by the decrease of the initial density of the granular soil skeleton.

3.3. Experimental investigation

The mechanical processes between the granular soil skeleton (Dembicki E., Kisielowa N. 229) and the fluid and gaseous pore contents during explosive soil loading play the essential role in the compaction process. Such mechanical processes have been observed in a study on a large-scale model experiment based on similarity theory. The large geometrical scale applied to the model tests made them comparable to small field tests.

In Figure 13, the schematic overview of the experimental device can be seen, which consists of an outer concrete and an inner wooden basin. The wooden basin is covered with geotextile on its inner surface that retains the model material applied. A free water-table can be simulated in the device at a desired height by filling the outer concrete basin with water.

The model experiments carried out consisted of the installation and saturation of the model material, observation of the initial state with zero measurements, initiation of the explosive charge, measurement of physical processes during explosive loading, and execution of control measurements on the compacted material. The experiment was finished with dewatering and removal of the model material from the experimental device. Explosion induced particle velocities and pore pressures have been registrated with real-time measurement systems. The particle velocity and pore pressure sensors have been placed directly into the model material. Additionally, time dependent settlements on the model surface were measured with a stereogrammetric monitoring system. Both fluid saturation and density were observed with radiometric measurements. Additionally, the determination of compaction effects was carried out on undisturbed samples taken from the model material before and after the explosion impact. To avoid difficulties at sampling in water-saturated soils, the undisturbed samples were taken before saturation and after dewatering, respectively.

A fine sand with a very narrow grain-size distribution and a mean grain size of dm=0.25 [mm] was used as model material in the experiments. The void ratio of dry model material has been measured at loosest and densest state in laboratory with emax=0.873 [1] and emin=0.560 [1], respectively. In the model material, an average initial dry density of pdi0=1.3675 |g/cm3] was reached during installation by dumping and slight pre-compaction with vibration. The obtained initial density corresponds to an average density index of ID0=-0.21 [1|, where the negative sign indicates the previously mentioned fact, that dumped wet granulates can reach a lower initial density than the density corresponding to the loosest grain packing state determined in laboratory on dry material.

The simulation of the water table level in a depth of hn=30 [cm] below the model surface has been reached by filling the outer concrete basin with water to the desired height. In the saturated model material an average initial saturation ratio of Sr=0.93 |1] has been determined.

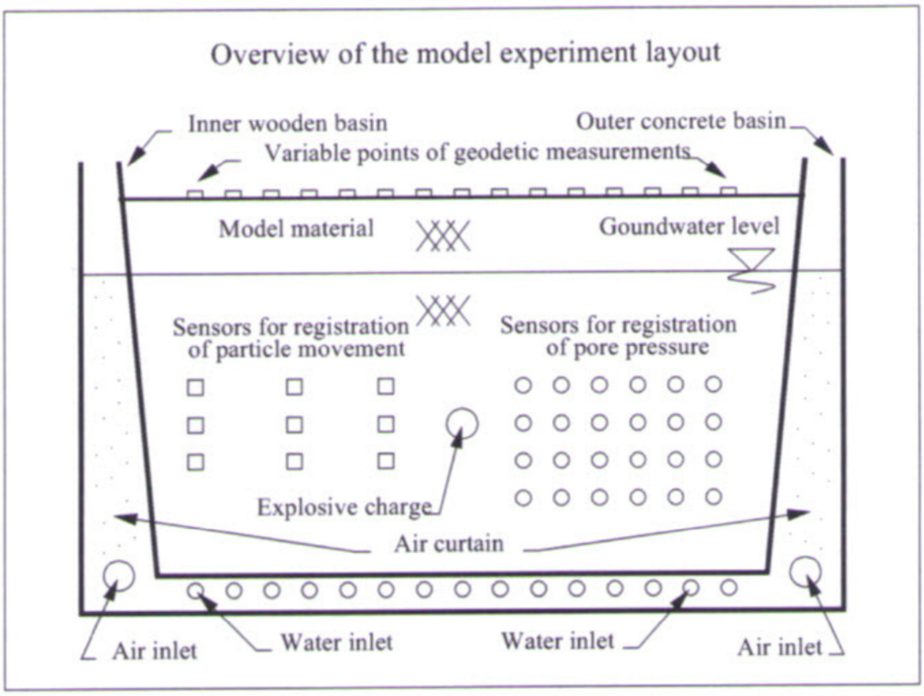


Figure13. Overview of the experimental device

The elaborated experimental results clearly point out, that firstly, the compaction energy is introduced into the soil in form of a high amplitude wave-field emerging from the explosive source. The pulsations of the cavity of gaseous explosive products increase the extension of the soil zone, where residual pore pressures are generated and effective stresses are reduced. The residual pore pressures cause a full or partial strength loss in the soil, allowing the granular skeleton to reorganize itself in a denser structure.

The compaction of the soil skeleton takes place, that can be observed in the form of settlements on the surface of the improvement site. The density increase during explosive compaction does not result from the compression of the soil phases under the high pressures in the neighbourhood of the explosive but from the mechanical interaction between the granular skeleton and the pore contents of the soil. In the model experiment, the explosive compaction effect was observed to consist in two fractions. A significant part of the registrated density change was observed to occur quickly after the dynamic excitation of the soil and without significant change of the residual pore pressures accumulated during the transient loading. The compaction effects resulted from a quick separation between soil skeleton and pore contents. The second fraction of settlements occurred during the regeneration of effective stresses and pore pressure dissipation. Additionally, the consolidation process of the mobilized soil layer is influenced by the deforming naturally saturated soil layer, which is unloaded due to the decrease of effective stresses and loss of strength in the soil zone influenced by the explosion (Dembicki E., Imiołek R., 610).

3.4 Classification

Soil compaction using the blasting method has two classifications:

- according to blasting charge shape
- according to blasting charge location

Blasting charges may be spherical or may have an elongated shape.

According to the blasting charge location, the explosions are classified into:

- surface explosions where the blasting charge is located on the ground surface (Figure 14)

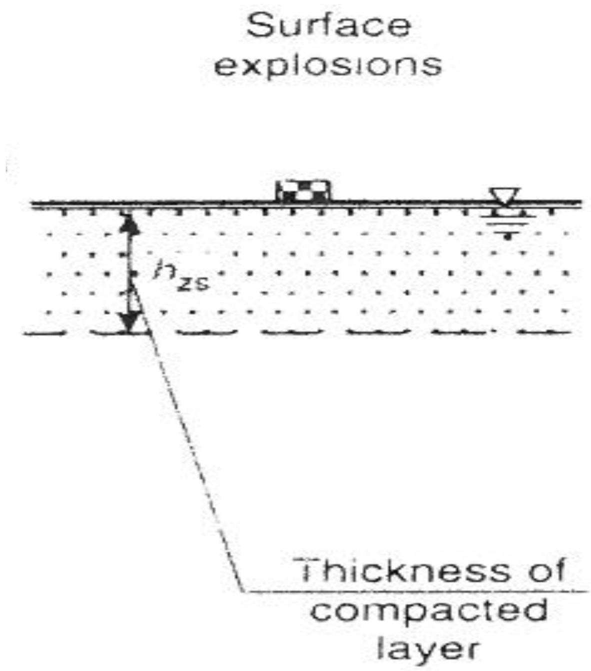


Figure 14. Surface explosions

- underwater explosions where the blasting charge is located underwater above the to-be compacted soil surface (Figure 15)

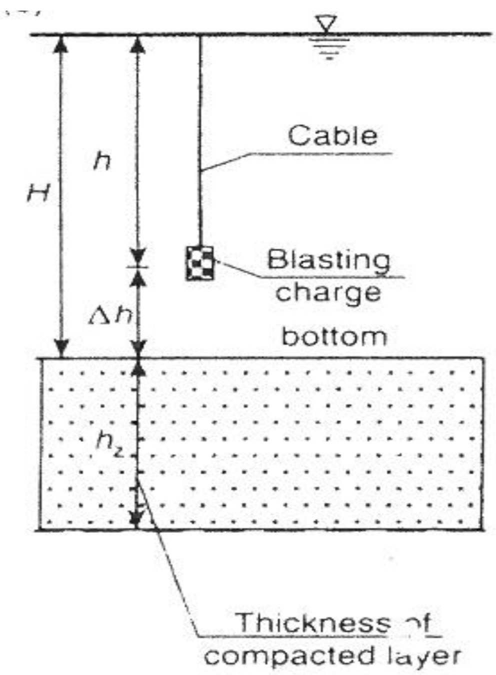


Figure 15.Underwater explosions

- hidden explosions where the blasting charge is located in the soil layer as a point, sectional or elongated charge (Figure 16 (c), (d), (e))

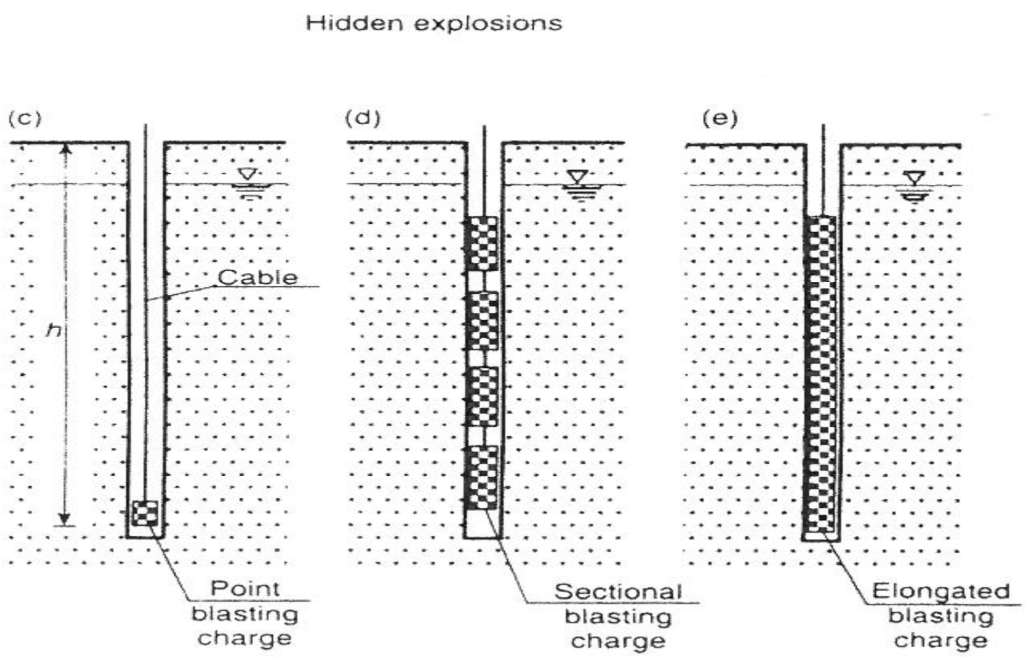


Figure 16. Hidden explosions

Here are the necessary formulas for practical calculation and for developing the technological design.

The charge mass, radius of effective charge action, thickness of compacted layer, height above the bottom at which the charge should be suspended and radius of sand drain can be determed. These formulas are based on the experience.

- For surface explosion

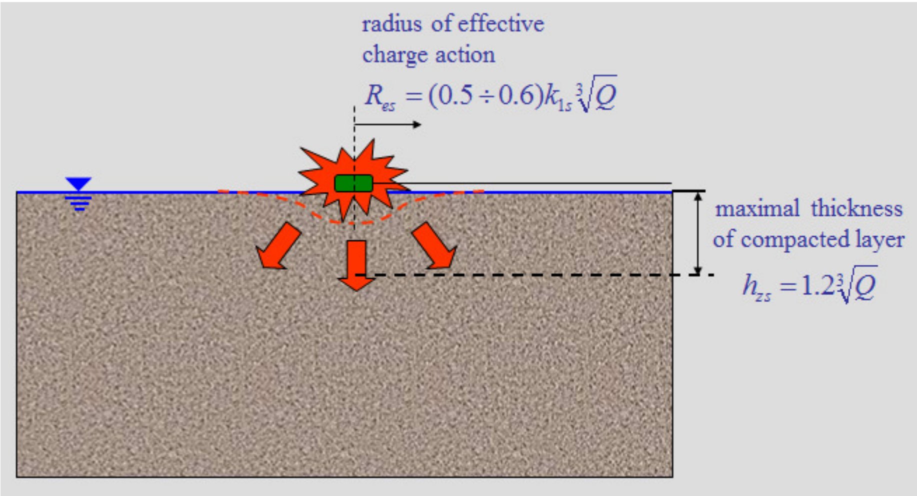


Figure 17. Radius of effective charge action for surface explosions

Radius R_{es}(m) of effective charge action

$$R_{es} = (0.5 \div 0.6)k_{13}\sqrt[3]{Q}$$ k_{13} = 2,5-3,0 (Ivanov, 1983)

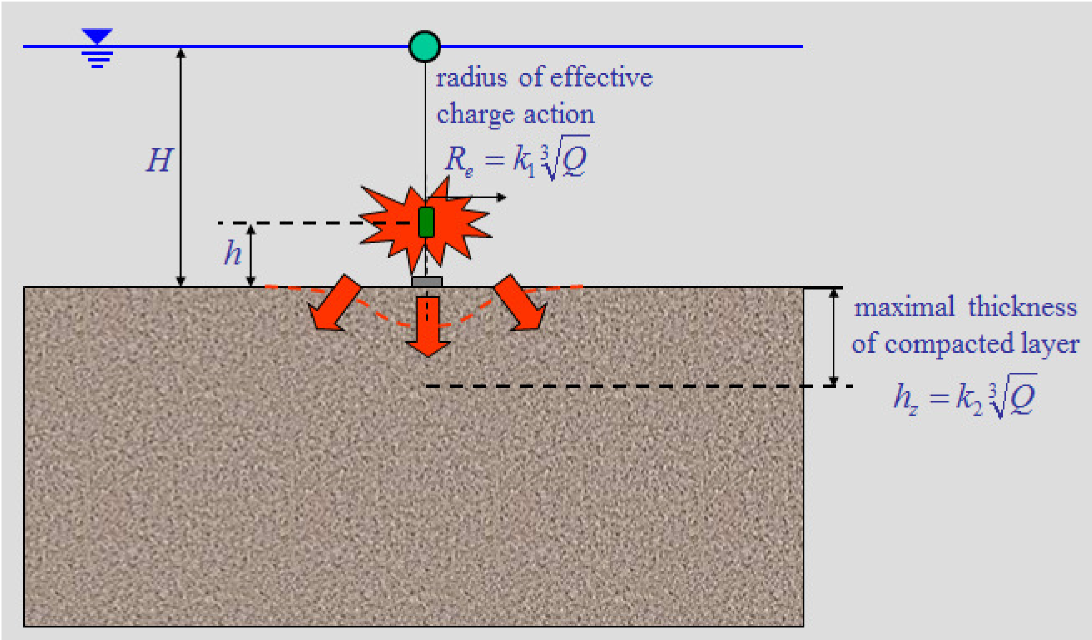
where Q (kg) is the mass of concentrated charge.

Thickness h_{zs} (m) of compacted layer

$$h_{zs} = 1.2k_{13}\sqrt[3]{Q}$$

Soil compaction using surface explosions is less effective than using underwater or hidden explosions

- For underwater explosions



- Figure 18. Radius of effective charge action for underwater explosions

Mass Q (kg) of concentrated charge

$$Q = 0.1H^{2.46}$$

where H (m) is depth of water.

Radius R_e (m) of effective charge action

$$R_e = k_1 \sqrt[3]{Q} \quad k_1 = 2,5\text{-}3,0$$

Thickness h_z (m) of compacted layer

$$h_z = k_2 \sqrt{Q} \quad k_2 = 4,5\text{-}5,0$$

Height Δ h (m) above the bottom at which the charge should be suspended

$$\Delta h = k_3 \left(\sqrt[3]{Q}\right)^{1.95} \quad k_3 = 0{,}35h$$

For hidden explosions and concentrated charges

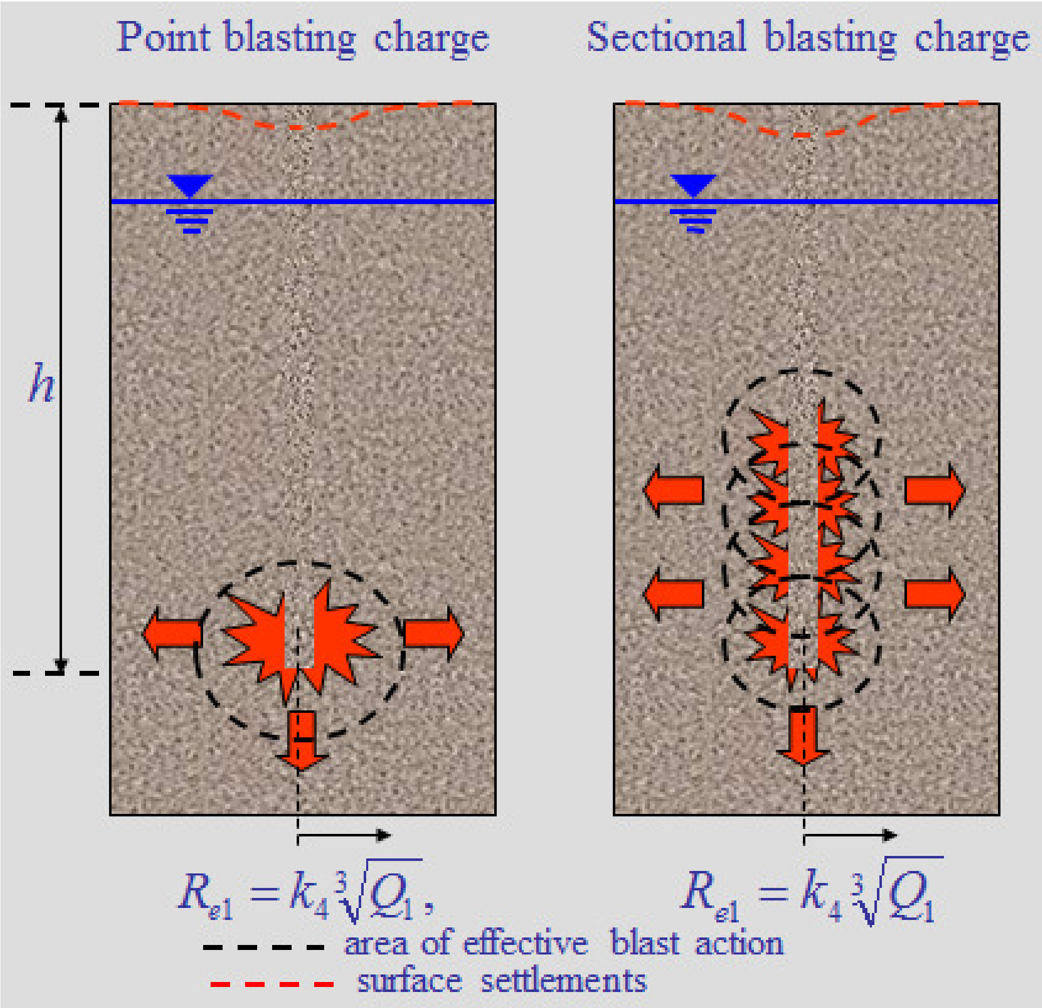


Figure 19. Radius of effective charge action for point and sectional blasting charges

Mass Q_1 (kg) of concentrated charge

$$Q_1 = 0{,}055h^3$$

where h (m) is the depth of the concentrated charge below ground surface.

Radius R_{e1} (m) of effective charge action

$$R_{e1} = k_4 \sqrt[3]{Q_1}, \;\; k_4 = 2.5 \div 3.0$$

For hidden explosions and elongated charges

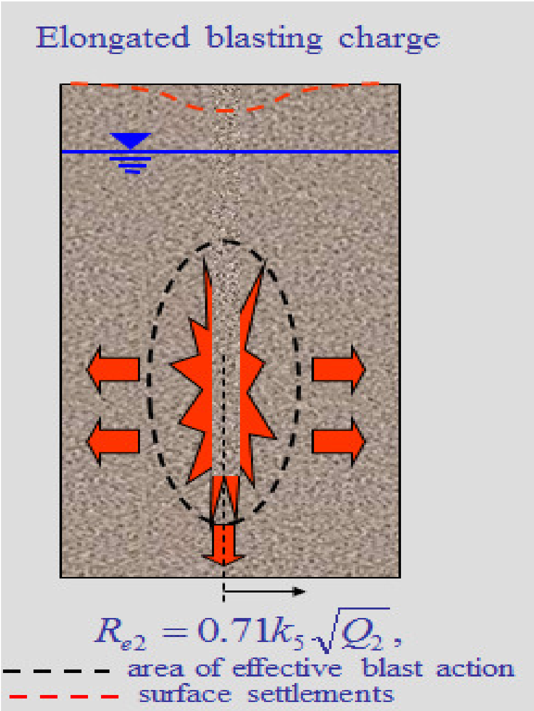


Figure 20. Radius of effective charge action for elongated blasting charge

Radius R,,2 (m) of effective charge action

$$R_{e2} = 0.71k_5\sqrt{Q_2} \quad k_5 = 2.5 \div 3.0$$

where Q_2 is unit length mass of blasting charge (in kg per metre).

Radius R_p, (m) of sand drain

$$R_p = k_p\sqrt{Q_2} \qquad k_p = 0.20 \div 0.35$$

Radius R_{ep} (m) of effective charge action for elongated blasting charge in cohesive soils (Dembicki E., Zadroga B., 21)

$$R_{ep} = k_6\sqrt{Q}, [m] \quad k_6 = 3.5 \div 4.5$$

3.5 Description of subgrade strengthening using microblasting technology

Dynamic subgrade strengthening with the help of microblasting technology includes the forming of columns which are made of a non-cohesive ground, accelerating the process of consolidation of weak organic soils (peat, silt, sapropel) or weak cohesive soils and also the compaction of loose non-cohesive soils.

The strengthening of subgrade begins with removal of vegetative soil layer and formation of a working platform with capacity not less than 1.5 m. Working platform creates favorable conditions for operation of construction equipment, and for the correct formation of the columns in the ground, which needs to be strengthened (in the case of weak organic or cohesive soils). The process of drilling boreholes from the level of working platform and carrying out of blasting operations must be in accordance with the following steps:

a) the identification of borehole locations on the building site with the help of the geodesic equipment in accordance with the project works;

b) the drilling of vertical boreholes without the protection of the pipe to the required depth of strengthening (Figure 21);

c) the checking of the depth of boreholes by the manager of the group carrying out the drilling, it is possible to increase the depth of the borehole to the base layer, which is determined by the resistance of the drill, and also macroscopic observation of excavate soil;

d) monitoring the continuity of the borehole by lowering the pole and measure its penetration;

e) the connection of the detonator and explosives (Figure 22);

f) moving of the explosive to the borehole (Figure 23);

g) the initiation of an explosive;

h) repeating these steps for each of the following boreholes.

As a result of the explosion of the charge in weak organic or cohesive soils column-drains are formed. These columns consist of a disconnected ground, the mission of which is to accelerate the consolidation process, and in the case of the loose non-cohesive soils the main target is the compaction of this soil.

To ensure the right way of the column- drains work boreholes need to be sunk to 0.5 m in the base layer (if the base layer consists of non- cohesive soils with high or medium density). The diameter of the shaped column- drains depends on the lateral resistance of the weak soil, and also on the unit mass of the charge. Due to the type of the process of columns formation a local discontinuity of the column can be formed and layers of organic soils or cohesive soils in the columns of loose material may be present. This phenomenon is not substantially affect to the process of strengthening the subsoil.

It is allowed to carry out works associated with the strengthening of subgrade by means of technology implosions without the formation of a working platform when there is non-cohesive soil with thickness greater than 1.0 meter in the surface layers.
After the blasting process it is necessary to align and seal the surface of working level by vibrating rollers (the process used in the case of future using of a working platform as the layer of the road embankment). On the ground base prepared in such a way, geomatras should be installed at the foot of the embankment. The geomatras is characterized by a special construction.

In the case of blasting in the areas with the presence of underground utilities all conflicts must be disclosed, and their location recorded. The decision of the future scope and method of blasting operations shall be made after assessment of the underground utilities in the joint consent of designers and engineers.

There is the decision to reduce the amount of gain subgrade in the area of conflict. It is approved by designers and engineers. In some cases, there may be the removal of areas of work to strengthen the subgrade with the help of implosions technology. In this case, in such zones it should be allowed to use another technology to strengthen the subgrade.


Figure 21. Drilling operations

Figure 22. Installation of a detonator

Figure 23. Moving of the explosive to the borehole

3.6 Practical consideration

Like many other geotechnical processes, explosive compaction has been designed largely on experience rather than theory.

Vibrations on nearby structures need to be controlled where blast densification is carried out. Blasting within 30-40 m of existing structures requires a reduction in the charge weights per deck and in the number of holes detonated at any one time. Also, when blasting is carried out adjacent to slopes, blast patterns are adjusted to restrict the zone of residual pore water pressure build-up and minimize the risk of slope instability.

For the above reasons, the number of charges detonated sequentially is often restricted to minimize the duration of shaking. Design of the appropriate charge delays between adjacent boreholes is carried out using the following process:

- Ground vibration patterns (peak particle velocities and frequency content) are determined at a particular location of concern remote from the blast point due to a single charge. This is best done using field measurements, but can also be carried out theoretically.
- The frequency range of potentially damaging vibrations is selected based on structural vibration theory or other considerations.

During all field work, appropriate seismic, geodetic or geotechnical measurements were carried out to control the effectiveness of soil compaction by explosions.

Seismic measurements are made with seismographs in order to determine the following factors:

- peak velocity and frequency of soil vibrations at various distances from the explosion center
- danger zone for charges of various mass, placed at different depths under the ground surface
- safe mass of charges for a given distance of explosion center from a protected object.

Geodetic measurements, consisting in leveling of the surface and deep helical benchmarks are made in order to determine:

- settlement of the surface and various subsoil layers at different depths
- changes of groundwater levels
- location of blasting charge in boreholes in plan, and the arrangement of detonation sleeves in boreholes
- volume and shape of craters formed after explosions.

Geotechnical measurements using various kinds of soundings are made in order to determine the state of soil compaction in horizontal and vertical planes, within the compacted area.

The measurements mentioned above were carried out before explosions (reference level), directly after each series of explosions, during the consolidation process, and after it was finished. The measurement work usually covered a period of one to two months.

3.7 Quality control of work

The range of quality control in the activities of the soil strengthening by microblasting method consists of:

- Measurements of the borehole length of drill penetration and its compliance with the length adopted in the work production plan, one measurement per 50 boreholes.
- Geodetic measurements of surface benchmarks - benchmarks are installed at intervals of 50 m (three series: one in the axis and two on each side of the plot)(33 b); surface benchmarks must be installed to the working platform. The initial measurement should be done before blasting, and the next measurement of settlement over 1 day after the end of this series of explosions, following measurements must be performed every three days for 4 weeks, until the difference between the average settlements of individual measurements is less than 2 cm (Ivanov P.L., 253).

4. Soil compaction at the base of road embankment in Odincovo district

4.1 Geological and engineering-geological processes

- Evaluation of the area with regard to karst-suffosion environment

In order to discover karst-suffosion environment on the site the existing regulatory documents, library materials, a map of engineering-geological zones of the city of Moscow and library materials, conducted area survey have studied, analyzed the results of these studies.

In the process of the engineering-geological studies boreholes with depth up to 30.0 m have been drilled. Based on the materials received the following engineering-geological features of the site have been discovered:

1. The upper part of the section is composed of sandy and clay quaternary deposits.
2. Hydrogeological conditions are characterized by post-Jurassic water table extent.
3. In the process of the rout area study funnel types of topography related to the karst-suffosion processes were not discovered on the surface.

In compliance with “Working map of engineering-geological zoning of the city of Moscow according to a hazard level of karst-suffosion processes occurrence” (Year 1996) this site is referred to the Zone I (safe with regard to karst-suffosion processes).

In general, under the existing geological structure and hydrogeological conditions and in accordance with the “Guidelines on structural engineering in areas of the city of Moscow where karst-suffosion processes occur” (Moscow, Year 1984) the site is tentatively estimated as not dangerous with regard to karst-suffosion processes.

- Evaluation of the area with regard to the possibility of slope processes development

On the territory of planned construction erosion slopes of the Sominka river valley are located. At the surface of slopes processes of sheet and linear erosion, as well as of deluvial run-off occur. Soil slips in the process of mapping were not discovered.

- Engineering-geological and hydrogeological conditions of building site

A site of highway construction and bridge across the Sominka river (Sta.99+50 – Sta.113+80) of the new exit to Moscow MKAD Ring Road from federal highway M-1 “Belorussia” Moscow-Minsk is situated in Odintsovo district Moscow region.

With regard to geomorphology the works site is confined to a fluvioglacial plain complicated by cutting of the valley of the Sominka river. Area absolute elevations are from 166.7 to 192.35 m.

Engineering-geological section structure at the depth explored is composed of quaternary deposits only. The modern technogenic mineral formations (tQ_{IV}) are common only for built-up areas and are presented by loams of low-plastic and semi-solid consistency (EGE-0v), with thickness of 0.6m. Late quaternary deluvial loams (dQ_{III}), dark brown, heavy silt (EGE -6t,6p,6tg,6m) of solid to high-plastic consistency lay under top soils (EGE – 1).

Alluvial deposits (aQ_{III}) are spread in the valley of the Sominka river, they were discovered both under deluvial deposits and on the surface and are presented by low-peaty loamy mass, with the maximum explored thickness of 33.8 m (Hole 687). They are laid by facially replaceable layers of low-plastic light silt loam (EGE-9atg), heavy silt loam (EGE-9bp, 9btg, 9pm) of semi-solid to high-plastic consistency, by light sandy loam of low-plastic (EGE-11atg) and high-plastic (EGE-11am) consistency, by low-plastic (EGE-15atg) and high-plastic (EGE-15am) clay. Low-peaty mass is underlaid by alluvial fine sands (EGE-12nv) of medium density, saturated with water, their bottom was not tapped up in the hole.

On sides of the valley of the Sominka river under deluvial soils Late Quaternary fluvioglacial deposits (fQ_{III}) were discovered. They are presented by interbedding of fine sands (EGE– 19nv) of medium density; sandy loams with gravel up to 5%, of solid to liquid consistency (EGE-24t,24pl,24tk); light and heavy loams (EGE-22p, 22tg, 23tg) and (EGE-20t, 20p, 20tg, 21t, 21p, 21tg, 21m) with gravel up to 5%, of solid to high-plastic consistency.

Semi-solid moraine loams of Moscow stage of glaciation (gQ_{II}^{ms}) are presented in the form of lenses. Heavy sandy loam (EGE-27p) with chalkstone gravels up to 10%, red-cinnamonic, with pockets of sand and sandy loams. The thickness is up to 5.9 m.

Under Moscow moraine there were discovered spots of intermorainal fluvioglacial deposits of Middle Quaternary age (fQ_{II}) represented by fluid sandy loams with gravel up to 5% (EGE-34tk) and semi-solid clays (EGE-35p), light (EGE-31p) and heavy (EGE-30tg,30p) loams of low-plastic and semi-solid consistency.

On the left bank of the Sominka river, under fluvioglacial soils lacustrine-boggy (lQ_{II}) deposits of Middle Quaternary age were found, they are presented by low-peaty clays (EGE-35bm, 35btg, 35bp, 35bt) and clays with organic matters inclusions (EGE-35atg) of grey-black colour, of solid to high-plastic consistency, with thickness of more than 10.2 m.

Under fluvioglacial deposits in the holes, moraine loams of the Don stage of glaciation (gQ_{II}^{dns}) embedded in the form of rocks, heavy sandy (EGE-37t) and heavy silt (EGE-38p) with break stones up to 10% of sedimentary and magmatic origins, of solid and semi-solid consistency were discovered. Pockets of sand saturated with water and of fluid sandy loams were found throughout the section, pockets from 0.5 up to 5.0 sm, water inflow of low level.

Below through the section of the left bank of the river Sominka fluvioglacial deposits of Early Quaternary age (fQ_I) locally distributed were discovered. The maximum thickness explored is 13.0m. Lithological composition is from fine tight sand saturated with water (EGE-45nv) to heavy sandy loam (EGE-40t) of solid consistency; their bottom was not tapped up in the hole.

The groundwater level was marked at absolute elevation of 164.65 m (Hole 692,781) and 186.28 m (Hole 776). Waters are free flow and confined to fluvioglacial soils: interbeds of sand and sandy loams in loams, as well as to sands and sandy loams.

Water is the environment non-aggressive to concrete of any density.

- Bridge across the Sominka river Sta.106+21,86

The described bridge across the Sominka river is located at Station 106 + 21.86 and is made of cast reinforced concrete with dimensions 2(G-15.25+.75) m, number and sizes of spans are 1x25.26 m, length 27.32 m. Angle of crossing with the projected road is 90°. Absolute elevations within the construction change from 167.03 m up to 167.15m (Bearing 1). The geological structure of the valley of the Sominka river at the depth of 35.0m is composed of late quaternary deposits only.

The upper part of alluvium is presented generally by low-peaty clay (Bearing 1) and loamy (Bearing 2) mass composed of clay of high-plastic (EGE-15am) and low-plastic (EGE-15atg) consistency with lenses (up to 1.5 m, Hole 137) by sapric slightly wet peat (EGE-18) and heavy silt loam of high-plastic (EGE-9bm) and low-plastic (EGE-9btg) consistency, as well as by light sandy loam of high-plastic (EGE-11am) and low-plastic (EGE-11atg) consistency. Hole 137 from the surface discovered fibric wet peat (EGE-1t), thickness of 0.6m. Low-peat mass maximum thickness explored is 33.5 m (Hole 135).

The section ends with lying under low-peat mass fine sands of medium density with gravel up to 10% (EGE-12nv) saturated with water, their bottom was not tapped up in the hole.

Ground waters are spread directly from the ground surface. Water is the environment that is not aggressive to concrete of any density.

- Soil properties

Physical and mechanical soil properties have been studied by laboratory and field methods.

Cone penetration test probes were taken in 12 points at the depth of up to 20 m by continuous penetration of probe of type II with use of measuring equipment “PIKA-15N” (Refer to sheets 185-196 of plates).

In order to determine an angle of soils` internal friction and adhesion, physical and mechanical properties for incoherent section was performed, together with field cone penetration test up to the depth of 20 m.

Soil grading was determined by means of particle-size screening and wet analyses.Based on the field descriptions, the laboratory findings, the CPT charts 56 EGE were specified on the engineering-geological sections in accordance with GOST 25100-95 and GOST 20522-96:

Table1 - Characteristics of soils

№	№№ EGE	The type of soil GOST 25100-95	Geological index	The density of sand and consistency of clay soils	C	φ	E
					кПа	град.	Мпа
1	4п	Loam light sandy	dQIII	semi-solid consistency	33	23	25,0
2	6т	Loam heavy silt	dQIII	solid consistency	34	24	24,2
3	6п	Loam heavy silt	dQIII	semi-solid consistency	30	21	23,2
4	6тг	Loam heavy silt	dQIII	low-plastic consistency	23	21	13,7
5	6м	Loam heavy silt	dQIII	high-plastic consistency	20	20	10,1
6	7п	Loam light silt	dQIII	semi-solid consistency	31	24	21,7
7	9тг	Loam light silt	aQIII	low-plastic consistency	17	19	6,8
8	9атг	Loam light silt	aQIII	low-plastic consistency	26	16	9,1
9	9бп	Loam heavy silt	aQIII	semi-solid consistency	20	20	10,8
10	9бтг	Loam heavy silt	aQIII	low-plastic consistency	17	19	6,9
11	9бм	Loam heavy silt	aQIII	high-plastic	15	17	4,4

№	№№ EGE	The type of soil GOST 25100-95	Geological index	The density of sand and consistency of clay soils	C	φ	E
					кПа	град.	Мпа
				consistency			
12	11атг	Loam light sandy	aQIII	low-plastic consistency	25	22	16,8
13	11ам	Loam light sandy	aQIII	high-plastic consistency	20	20	10,1
14	12нв	Fine sand	aQIII	medium density		34	29,8
15	13нв	Medium-grained sand	aQIII	medium density	1	36	32,5
16	15атг	Clay	aQIII	low-plastic consistency	25	14	3,4
17	15ам	Clay	aQIII	high-plastic consistency	23	13	2,0
18	15м	Loam heavy silt	lQIII	high-plastic consistency	27	13	5,0
19	18мвл	Sapric peat	lQIII	slightly wet	13	8	0,2
20	19мн	Fine sand	fQIII	medium density	3	33	31,4
21	19нв	Fine sand	fQIII	medium density		31	25,5
22	20т	Loam heavy silt	fQIII	solid consistency	37	20	23,9
23	20п	Loam heavy silt	fQIII	semi-solid consistency	30	22	22,8
24	20тг	Loam heavy silt	fQIII	low-plastic consistency	28	23	18,9
25	21т	Loam heavy sandy	fQIII	solid consistency	46	25	26,8
26	21п	Loam heavy sandy	fQIII	semi-solid consistency	31	23	28,6
27	21тг	Loam heavy sandy	fQIII	low-plastic consistency	28	20	17,2
28	21м	Loam heavy sandy	fQIII	high-plastic	23	19	16,1

№	№№ EGE	The type of soil GOST 25100-95	Geological index	The density of sand and consistency of clay soils	C	φ	E
					кПа	град.	Мпа
				consistency			
29	22п	Loam light sandy	fQIII	semi-solid consistency	26	22	28,7
30	22тг	Loam light sandy	fQIII	low-plastic consistency	25	20	21,0
31	23тг	Loam light silt	fQIII	low-plastic consistency	25	18	17,8
32	24т	Sandy loam	fQIII	solid consistency	16	28	20,2
33	24пл	Sandy loam	fQIII	plastic consistency	16	28	22,2
34	24тк	Sandy loam	fQIII	liquid consistency	2	26	11,0
35	24ат	Sandy loam	fQIII	solid consistency	16	28	18,8
36	24атк	Gravel loam	fQIII	liquid consistency	2	26	11,0
37	27п	Loam heavy sandy	gQIIms	semi-solid consistency	54	21	35,6
38	30п	Loam heavy silt	fQII	semi-solid consistency	32	20	26,9
39	30тг	Loam heavy silt	fQII	low-plastic consistency	28	19	20,6
40	31п	Loam light sandy	fQII	semi-solid consistency	33	21	26,4
41	31тг	Loam light sandy	fQII	low-plastic consistency	28	20	22,2
42	32тк	Sandy loam	fQII	liquid consistency	2	26	11,0
43	34тк	Sandy loam	fQII	liquid consistency	2	26	11,0
44	35п	Clay	fQII	semi-solid consistency	38	19	24,0
45	35атг	Clay	lQII	low-plastic consistency	29	13	8,0

№	№№ EGE	The type of soil GOST 25100-95	Geological index	The density of sand and consistency of clay soils	C	φ	E
					кПа	град.	Мпа
46	35бт	Clay	lQII	solid consistency	48	14	9,3
47	35бп	Clay	lQII	semi-solid consistency	47	15	7,1
48	35бтг	Clay	lQII	low-plastic consistency	41	12	5,8
49	35бм	Clay	lQII	high-plastic consistency	31	9	4,5
50	37т	Loam heavy sandy	gQIdns	solid consistency	47	26	54,4
51	37п	Loam heavy sandy	gQIdns	semi-solid consistency	64	21	29,9
52	38п	Loam heavy silt	gQIdns	semi-solid consistency	70	23	30,4
53	40т	Loam heavy sandy	fQI	solid consistency	41	25	35,9
54	44т	Clay	fQI	solid consistency	52	18	31,3
55	45нв	Fine sand	fQI	tight	4	35	35,5
56	46п	Loam heavy silt	fQI	semi-solid consistency	33	24	29,2

4.2 Climatic conditions

The area is located within the bounds of the Smolensk-Moscow Upland on the Moskvoretsk ridge that separates the Moscow river basin from the basin of its Pakhra feeder. A Belorussian railroad at the first approximation runs just along this water-parting line. Absolute elevations of the water-parting line have marks at 220 m B.S. Spurs of this ridge are approaching the Moscow river being water-parting lines between its feeders. Their limit heights are from 192 to 205 m.

The area`s terrain of the projected road is aqueoglacial formed by local aqueoglacial streams, gentle hills are intersected by small shallows.

Within the Odintsovo district there are 412 stream flows. From 70 to 140 of them are rivers, others are temporary streams (creeks) in small valleys, hollows, and shallows. These are typical lowland rivers and all of them, except the Moscow river, are referred to the category of minor rivers and are of the basin of the Oka river. Water from the territory of the area comes into the Oka through its two feeders – the Moscow and the Nara. 86% of the area coverage belongs to the basin of the Moscow river, and 14% to the basin of the Nara. Flow of the Moscow and Nara rivers in their origin is safely regulated by water reservoirs. An important factor of flow control is a relatively high proportion of spring waters in river inflow, because the given area is located within the bounds of the Smolensk-Moscow Upland where the rivers flow along more deeply incised valleys than outside uplands. Significant elevation changes mean deep incut of valleys of minor rivers. They are “cutting through” the mass of sedimentary rocks to deep water-bearing stratum and in summer season are mainly fed by spring waters. Activity of spring waters does not change during a year that results in relative discharge uniformity of the minor rivers.

Stream flows intersected by the road are mainly fed by melt waters and precipitation. Maximum upwelling is observed after clearance of ice. Spring flood occurs at the late March – middle April. The highest waters correspond to levels of spring flood. A low-water level is at the end of May – the middle of June and usually remains till September. The first ice phenomena on the rivers of the area are generally registered in the second decade of November, 3-5 days after average air temperature crosses 0°C. Ice coverage usually falls at the end of November – the beginning of December. The thickest ice on the rivers is at the end of February – the beginning of March.

On snaky and shallow rivers that are stream flows on the area under study there is no ice and timber drift.

The climate is temperate continental; it is characterized by transient features of continental climate of the eastern regions of the European territory of the country to more humid climate of the north-western regions. The summer is relatively warm, the winter is moderately cold with stable snow cover and well-marked mid-seasons.

The main climatic characteristics based on data of the long-term records of weather stations that are the nearest to the alignment of the designed road: Moscow Hydrometeorological observatory, Novo-Jerusalem town and Nemchinovka village, published in the USSR scientific-engineering climate data sheet (Series 3. Parts 1-6. Issue 8. Moscow and the Moscow region) and SNiP 23-01-99 [5, b], are presented in tables 3.3.1-3.3.11, wind charts – on fig. 3.3.1, climatogram for Nemchinovka weather station – on fig. 3.3.2. In Supplement 2 the main climatic characteristics for weather stations Nemchinovka and Novo-Jerusalem averaged for a ten-year period from 1986 to 1995 are given.

Atmospheric circulation plays an important role in climate formation of the area under study. In cold seasons south, south-west and west winds are more often, they bring warm air from Atlantics with cloudy weather and rainfalls. Therefore winter here is not severe.

In summer a wind direction changes, the proportion of north winds is rising, the winds provide cool weather in warm seasons. Continental air of temperate latitudes provides warm weather with variable clouds, showers, and thunderstorms in summer. In winter this air forms moderately freezing weather without precipitation.

Inflow of cold arctic air from regions of the north seas causes cold and night frosts in spring. In summer it causes cloudy cold weather without precipitation and in winter clear cloudless freezing weather. Sometimes cyclones come from the western areas, in autumn and winter it happens more often. In this case there are thick clouds, rainfalls, and strong gusty winds.

In average during the year south-western and western winds are prevailing (Table 3.3.8, fig.3.3.1). An average annual wind speed is 2.8-3.6 m/sec. An average annual number of days with strong wind (15 m/sec. and more) equals from 2 to 16 days in various places, the most is up to 32 days (w.s. Moscow Hydrometeorological observatory).

An average annual air temperature is from 3.4°C to 4.1°C. The coldest month is January, with an average monthly temperature of minus 10.2-10.8°C, the warmest month is July with an average monthly temperature of 17.5-18.1°C. In warmest years an average monthly air temperature in July reaches 22-24°C. In cold winters an average monthly air temperature in January-February falls to minus 14-15°C. The air temperature absolute minimum is minus 53°C (w.s. Novo-Jerusalem). The air temperature absolute maximum registered in July and August is 36-37°C.

Relative air humidity changes during the year over a wide range. During the year the highest air humidity is registered in December and January, it complies with the minimum air temperature (Table 3.3.4). In cold seasons relative humidity slightly changes within the territory and in December-February it is 84-87 %. Starting from March humidity starts lowering and reaches its minimum in May. During the year atmospheric precipitations minimum is registered from February to April, and the maximum falls on July (Table 3.3.7). In average during the year there are 13-18% of solid precipitations, 68-75% of liquid and 12-15% of mixed ones (slush, snow with rain).

Duration of the winter period is 4-5 months. The first autumn frosts are registered in the middle-at the end of September. In last decade of November stable frosts start and last till the middle of March. The last frosts can be registered up to the 20-th of May, and sometimes even in the first decade of June. In average during the year stable frosts last for 105-113 days. In winter the soil is freezed through at the depth of 56-75cm. In cold winters the depth of soil freezing increases up to 120 cm (Table 3.3.12). Snow coverage height plays a significant role here. In winter with little snow the soil is freezed deeper. An average decade snow coverage height lies within limits from 30 to 53 cm, the maximum height is 52-83cm, and in winters with little snow, snow coverage height is not more than 17cm. Snow cover appears about October 28-31, and disappears on April 9-15. It lies in the area of the road for about 139-142 days.

The area of the designed road lies in Climatic zone II for road building in the RF on the territory of the Moscow region that is referred (according to the Industrial Road Norms 218.014-99) to the regions of medium snow control difficulty.

Thaws are registered even in cold winters. The most days of thaw may be in November and March – for 15-17 days, and in December, January and February – for 5-8 days.

In winter, mainly from December to February, about 28-64 days with snowstorms and the same number of days with surface icing are registered.

Atmospheric phenomena: - an average annual number of days with mists is 26-37, the maximum number of days is 49-74;

- an average annual number of days with thunderstorms is 23-27, the maximum number of days is 34-41;

- an average annual number of days with hail – 1.7-1.9, the maximum number of days is 4-8.

Number of days with possible cases of winter slippery formation (based on data of “Guidelines on measures to control winter slippery on automobile roads”, approved by the Ordinance of the Ministry of Transport of Russia No. OS-548-r dated June 16, 2003) – number of days with snowfall with daily amount of more than 1cm (from light snow to abundant snowfall), with icing, hoarfrosting phenomena (slush, hoarfrost, icy rain) with air temperature lower than 0^oC is 79 days. The mean date of the beginning of the period with possible cases of winter slippery formation is the 5-th of November, of the end of the period is the 5-th of April, period duration is 152 days.

4.3 Description of the project

Due to the complex geological conditions, it was decided to strengthen the soil on the sections:

PC 105+ 70 - PC 108+70

- Setup of project

The following design settings were taken for these sections of subgrade strengthening:

the mark of the planned road - **175,17 ÷ 179,56 m**
the height of the embankment - **0,21 ÷ 9,13 m**
the mark of territory before removing the layer of plant soil - **166,96 ÷ 179,35 m**
the mark of the groundwater level - **0,00 ÷ - 0,20 m above the ground surface**

It is supposed to remove the top soil (the layer of plant soil) and form the working platform with the height of 1.5-3 meters. We need to use a disconnected ground (ground that is used for the construction of the embankment) to create a working platform. The filtration coefficient of such soil is more than 6,0 m/day (k≥6,0 m/day), that provides the required compaction achievement.

- The calculation of settlements

Engineering-geological conditions are essentially uniform. Therefore, any calculations in this case will be estimated. It is reasonable to consider boreholes, which are located in soils with the most different engineering-geological conditions. Such boreholes are №166 and №687, a distance between them is 26.4 m.
To assign the coefficient of permeability the reference table is used.

Table 2 - permeability coefficients of soils

Soil	permeability coefficient,m/day
Coarse sand	25 – 75
Medium-grained sand	10 – 25
Fine sands	2 – 10
Silt sand	0,1 – 2
Sandy loam	0,1 – 0,7
Loam	0,005 – 0,4
Clay	0,005

First of all, we need to estimate the time consolidation of the base embankment by the simpliest method of engineering. It is necessary to estimate the approximate time consolidation for the stratification of soil for boreholes №166 and №687

The average coefficient of permeability for hole № 166 is 0.01 m/day, for hole № 687 0.005 m/day. The average modulus of deformation is 8 MPa. The capacity of clay soil layers for hole № 166 is 17,1 m, for hole № 687 is 33.8 m.

Poisson ratio = 0,3.

Coefficient β=1-2*0.3^2/(1-0.3)=0.743.

the coefficient of consolidation for hole № 166

0.01/(0.743/8000*10)=10.8 m^2/day,

for hole № 687

0.005/(0.743/8000*10)=5.4 m^2/day,

for hole № 166 N=π2*10.8/(4*17.1^2)*t= 0.09*t.

for hole № 687 N=π2*5.4/(4*33.8^2)*t= 0.01*t.

Calculation of the consolidation degree perform in accordance with the expression

U=1-8/ π2*(exp(-N)+1/9*exp(-9N)+…)

Dependence degree of consolidation from the time (Figure 24).

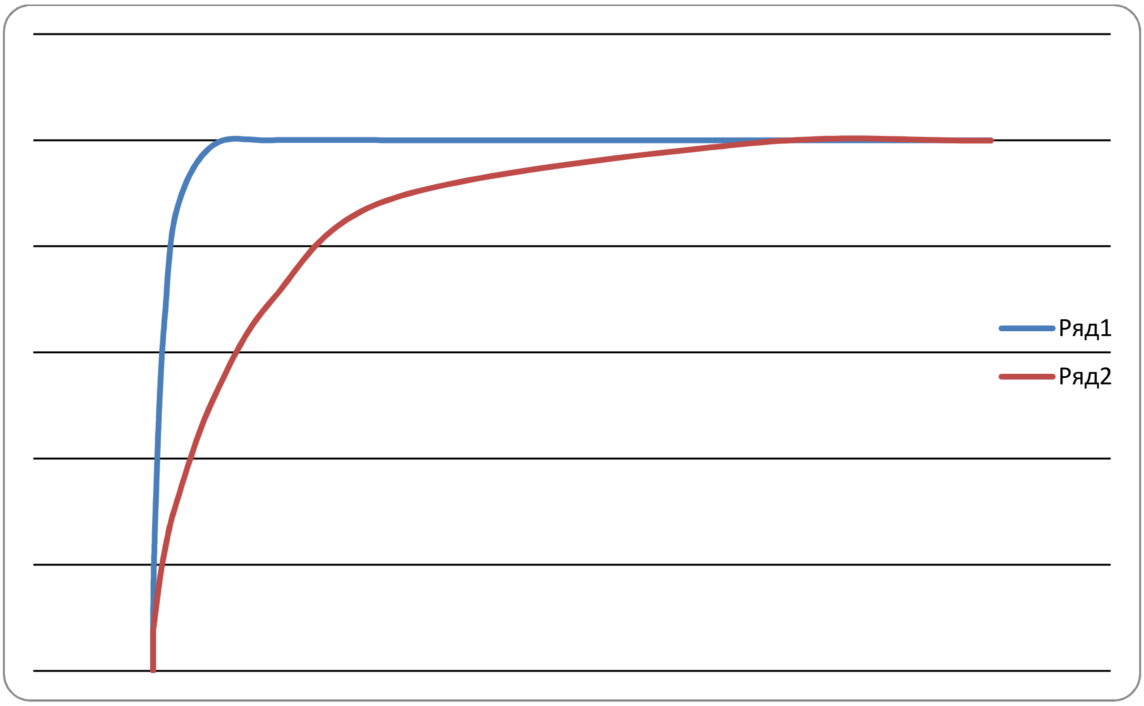


Figure 24. Dependence degree of consolidation from the time (days) Blue - the calculation for hole № 166, red - for hole № 687.

The graph shows that the approximate time of the consolidation of the embankment at the base for hole № 166 is about 50 days, for hole № 687 about 1,5 years.

It shows that the base of the mound at the site is uniform. Based on the significant difference of consolidation time and settlements of the bases on a small area by the length of the embankment, it is impractical to organize an embankment on a natural basis without any artificial measures to improve the base. In this regard, the using of sand piles-drains to strengthen the soil drains was to consider. Their diameter is 600 mm. The piles are located on the grid of 5 x 5 metres.

- The details of the project

At the design stage it was decided to strengthen the soil with the pile drains. It was planned to drill boreholes with the diameter of 0.6 meters.

The volume of the subgrade strengthening

The volume of the subgrade strengthening by microblasting method includes the following sides:

PC 105+70 – PC 108+70

- Sections of strengthening: 1-15
- The depth of subgrade strengthening: 11-25,5 m.
- The quantity of boreholes: 973
- The total length of drilling: 23217,5 r m.

The amount of the subgrade strengthening

The subgrade strengthening involves the implementation of boreholes in a square grid with line space of 5 x 5 m and includes the following sections:

PC 105+70 - PC 108+70

- Section 1
  → The depth of subgrade strengthening: 11 m
  → The depth of drilling: 12,5 m
  → The number of boreholes: 26 m
  → The total length of boreholes: 325 m
- Section 2
  → The depth of subgrade strengthening: 12 m
  → The depth of drilling: 13,5 m
  → The number of boreholes: 29 m
  → The total length of boreholes: 391,5 m
- Section 3

→ The depth of subgrade strengthening: 12,5 m
→ The depth of drilling: 15,5 m
→ The number of boreholes: 67 m
→ The total length of boreholes: 1038,5 m

- Section 4

→ The depth of subgrade strengthening: 15,5 m
→ The depth of drilling: 17 m
→ The number of boreholes: 51 m
→ The total length of boreholes: 867 m

- Section 5

→ The depth of subgrade strengthening: 17 m
→ The depth of drilling: 18,5 m
→ The number of boreholes: 61 m
→ The total length of boreholes: 1128,5 m

- Section 6

→ The depth of subgrade strengthening: 20,5 m
→ The depth of drilling: 22 m
→ The number of boreholes: 92 m

→ The total length of boreholes: 2024 m

- Section 7

→ The depth of subgrade strengthening: 25 m
→ The depth of drilling: 26,5 m
→ The number of boreholes: 290 m
→ The total length of boreholes: 7685 m

- Section 8

→ The depth of subgrade strengthening: 26 m
→ The depth of drilling: 27,5 m
→ The number of boreholes: 34 m
→ The total length of boreholes: 935m

- Section 9

→ The depth of subgrade strengthening: 33 m
→ The depth of drilling: 34,5 m
→ The number of boreholes: 34 m
→ The total length of boreholes: 1173 m

- Section 10

→ The depth of subgrade strengthening: 34,5 m
→ The depth of drilling: 36 m
→ The number of boreholes: 85 m
→ The total length of boreholes: 3060m

- Section 11

→ The depth of subgrade strengthening: 31,5 m
→ The depth of drilling: 33 m
→ The number of boreholes: 34 m
→ The total length of boreholes: 1122m

- Section 12

→ The depth of subgrade strengthening: 25,5 m
→ The depth of drilling: 27 m
→ The number of boreholes: 34 m
→ The total length of boreholes: 918m

- Section 13

→ The depth of subgrade strengthening: 22,5 m
→ The depth of drilling: 24 m
→ The number of boreholes: 34 m
→ The total length of boreholes: 816m

- Section 14

→ The depth of subgrade strengthening: 16,5 m
→ The depth of drilling: 18 m
→ The number of boreholes: 34 m

→ The total length of boreholes: 612m

- Section 15

→ The depth of subgrade strengthening: 15 m
→ The depth of drilling: 16,5 m
→ The number of boreholes: 68 m
→ The total length of boreholes: 1122m

The strengthening of subgrade with the help of microblasting technology includes the formation of 973 boreholes, the total length of which is 23217,5 m.
Appendix 1, Appendix 2 and Appendix 3 show the plan, cross-section and longitudinal profile of this project.

4.4 Comparison of settlements and time of consolidation

Preliminary analysis was carried out and the time of consolidation and settlements from the influence of soil weight of the road embankment are determed. In the first case, a calculation was performed for the unreinforced embankment. In the second case, the calculation was made for the strengthened embankment using the pile-drains. It is useful to consider a site with the deepest occurrences of weak soils. Such cross-section is shown in Figure 25.

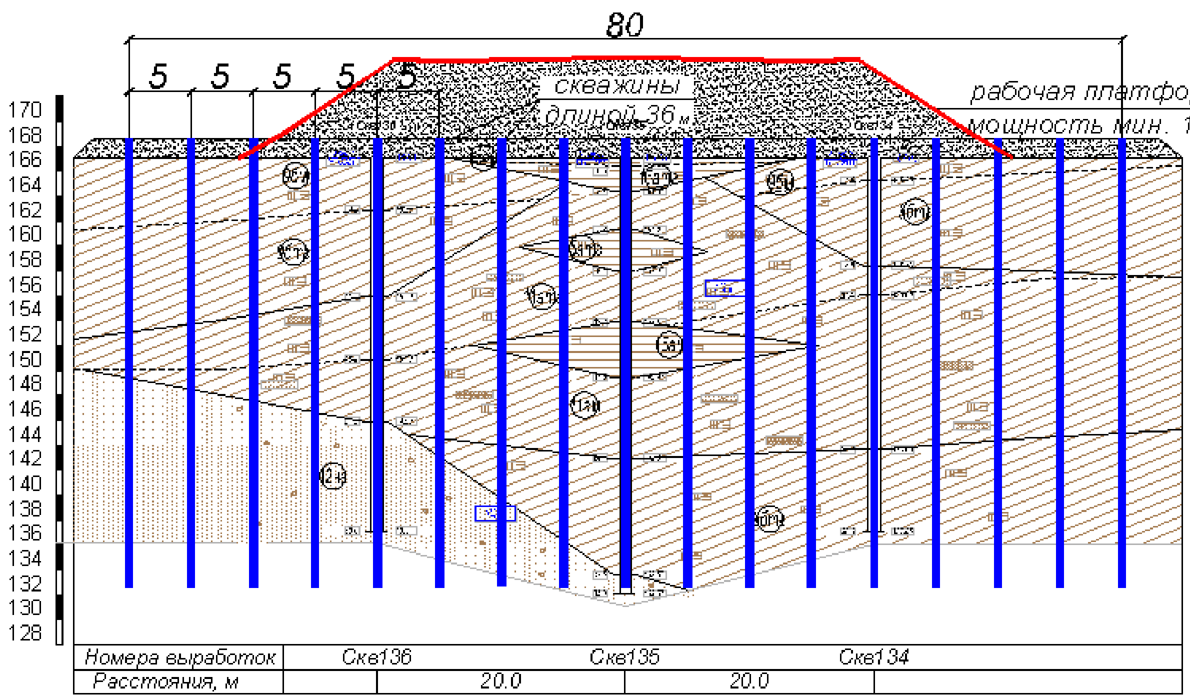


Figure 25. The cross-section of the embankment (PC106 + 30)

Alluvial deposits are typical soils at the PC 106+30.

Alluvial deposits (aQ_{III}) are spread in the valley of the Sominka river, they were discovered both under deluvial deposits and on the surface and are presented by low-peaty loamy mass, with the maximum thickness explored of 33.8 m (Hole 687). They are laid by facially replaceable layers of low-plastic light silt loam (EGE-9atg), heavy silt loam (EGE-9bp, 9btg, 9pm) of semi-solid to high-plastic consistency, by light sandy loam of low-plastic (EGE-11atg) and high-plastic (EGE-11am) consistency, by low-plastic (EGE-15atg) and high-plastic (EGE-15am) clay. Low-peaty mass is underlaid by alluvial fine sands (EGE-12nv) of medium density, saturated with water, their bottom was not tapped up in the hole. Soil characteristics are presented in Table 3.

Table 3 – soil characteristics

№	№№ EGE	The type of soil GOST 25100-95	Geological index	The density of sand and consistency of clay soils	c, кPa	φ,	E, MPa
1	9btg	Loam heavy silt	aQIII	low-plastic consistency	17	19	6,9
2	9bm	Loam heavy silt	aQIII	high-plastic consistency	15	17	4,4
3	11atg	Loam light sandy	aQIII	low-plastic consistency	25	22	16,8
4	11am	Loam light sandy	aQIII	high-plastic consistency	20	20	10,1
5	12nv	Fine sand	aQIII	medium density	0	34	29,8
6	15atg	Clay	aQIII	low-plastic consistency	25	14	3,4
7	15am	Clay	aQIII	high-plastic consistency	23	13	2,0

The calculation of consolidation was performed using program complex Plaxis 2D. Elastoplastic Mohr-Coulomb model was used as an ideal soil model.

The calculation was performed for two tasks:

1) Calculation of consolidation without the use of pile- drains (Figure 27)
2) Calculation of consolidation with the pile-drains (Figure 28)

Figure 26 shows the deformation situation for a task, when driving drains are deactivated. Settlement of the embankment is 636 mm, and the time of consolidation for achieving the minimum pore pressure is 64 days (Minimum pore pressure is 1kH/m2).

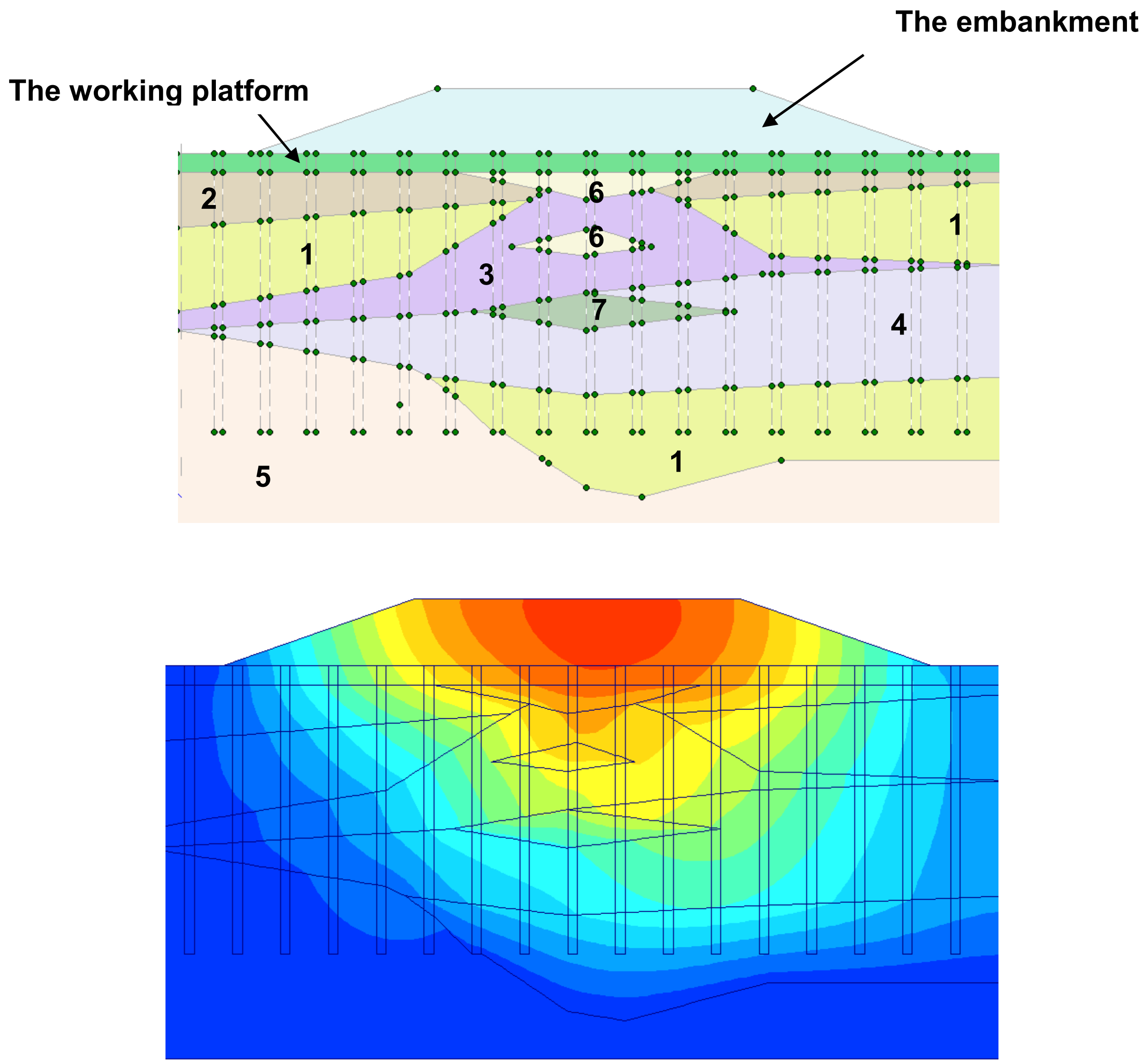


Figure 26. The model and the picture of deformations.

When pile-drains (made by drilling) were used the settlement is 525 mm and consolidation time is reduced to 37 days.

For the first task

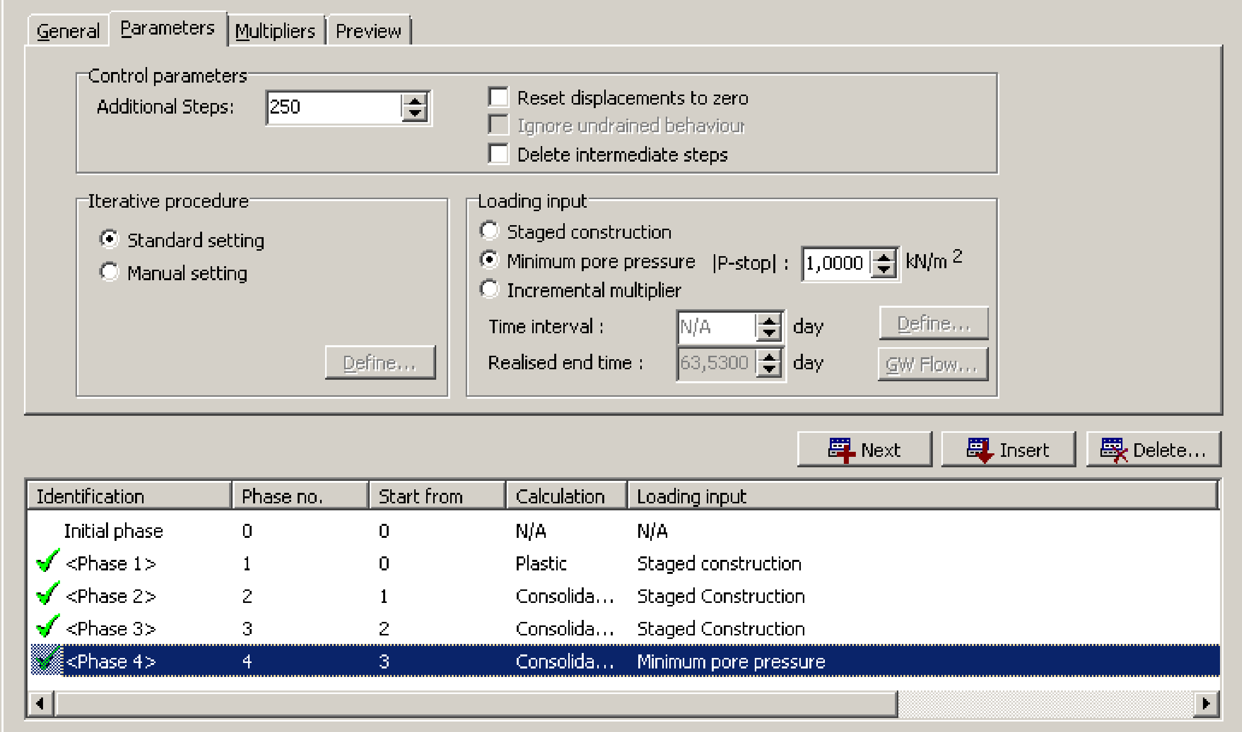


Figure 27

For the second task

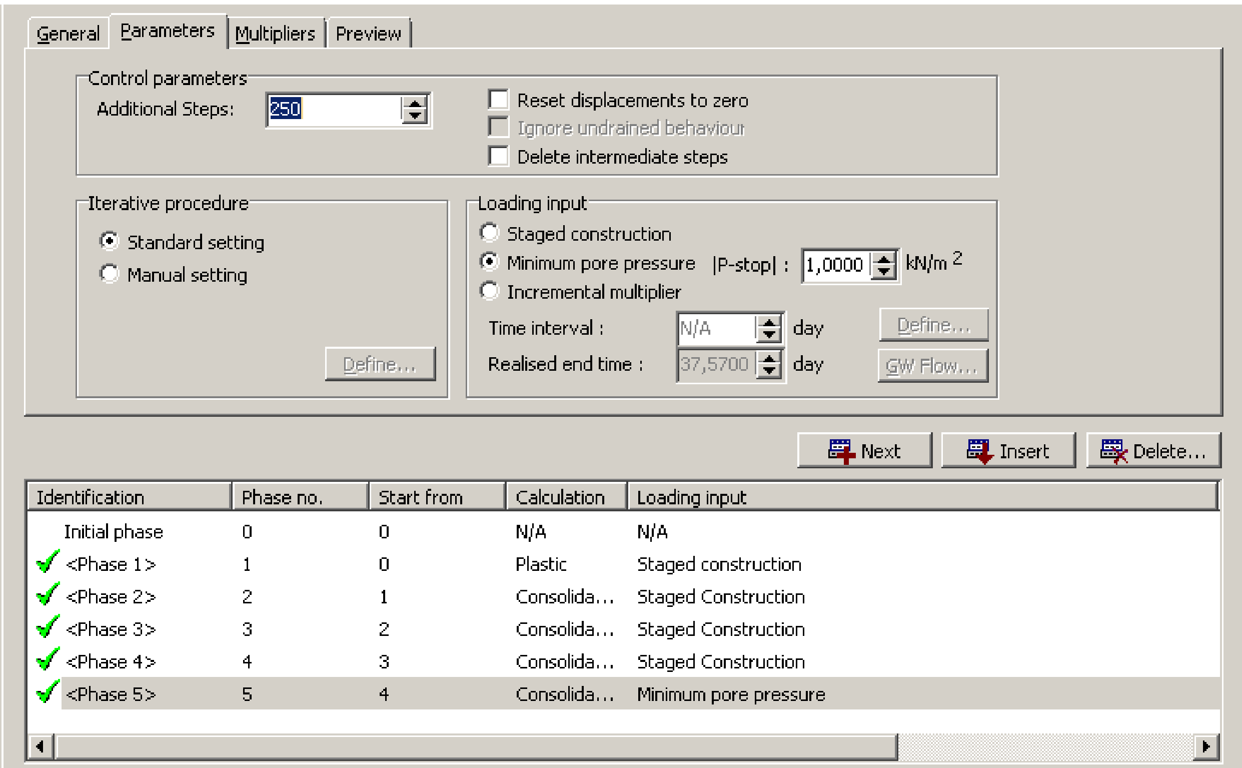


Figure 28

On a stage of working draft it was decided to form pile-drains by microblasting method.

The reasons for this decision were the following factors:

→ economic efficiency
→ speed of execution of works
→ high rate of consolidation

- Adoption of the design decisions

Based on initial data the following design decisions are determined:
Line space of boreholes square grid is 5 x 5 m

Radius R_{ep} (m) of effective charge action for elongated blasting charge in cohesive soils

$$R_{ep} = k_6\sqrt{Q}, [m] \quad k_6 = 3.5 \div 4.5,$$

where Q is unit length mass of blasting charge (in kg per meter).

$R_{req} = 3{,}54\ m$ - the minimum required radius of ground strengthening in cases when line space of boreholes square grid is 5 x 5 m.

$$R_{ep} = 4{,}0\sqrt{1{,}0} = 4{,}0 \geq 3{,}54\ m$$

The estimated radius of the sand columns-drains formed by the implosions

$$R_p = k_p\sqrt{Q_2}, [m] \quad k_p=0{,}20 \div 0{,}35$$

where Q_2 is unit length mass of blasting charge (in kg per meter).

$$R_p = 0{,}30\sqrt{1{,}0}, = 0{,}30\ m$$

The estimated radius of the sand columns-drains is about 0,60 meters.

a) It is expected that the implementation of boreholes and the dynamic consolidation of cohesive soils will achieve to an average depth of 14.77 m;
b) Before the blasting process it is necessary to establish benchmarks surface. The measurements of settlements will be made for at least four weeks.
c) After the process of subgrade improvement, the working platform will be leveled and will be a part of a road embankment

The volume and the amount of the subgrade strengthening are the same with the first project, where pile-drains were made without microblasting.

- Settlements and time of consolidation

  The distribution of the settlements after base strengthening and also after embankment construction were approximately determined. The cross-section for the most unfavorable geological conditions was used for the calculations. It is borehole № 135 (PC 106+40). The height of the embankment is 8.09 m at this cross-section.

  The soil parameters are used:

Table 4

Parametrs of soils – mean value							
The layer		15am	15atg	11atg	11am	9btg	12nv
Type of ground		I(+T)(mpl)	I+T(pl)	Gp//P(pl)	Gp//P(mpl)	Gπ(+T)(pl)	Pd(szg)
parameter	unit	**value**	**value**	**value**	**value**	**value**	**value**
$\square^{\square n)}$	[º]	13,00	14,00	22,00	20,00	19,00	34,00
$c^{\square n)}$	[kPa]	23,0	25,0	25,0	20,0	17,0	
□	[-]	0,42	0,42	0,35	0,35	0,35	0,30
□	[-]	0,39	0,39	0,62	0,62	0,62	0,74
$M_o^{(n)}$	[MPa]	5,11	8,68	26,96	16,21	11,07	40,12
$E_o^{(n)}$	[MPa]	2,00	3,40	16,80	10,10	6,90	29,80
$\square_s$	[g/sm^3]	2,74	2,74	2,71	2,71	2,71	2,66
$\square_s$	[kN/m^3]	26,88	26,88	26,59	26,59	26,59	26,09
□	[g/sm^3]	1,68	1,75	1,74	1,72	1,82	2,03
□	[kN/m^3]	16,48	17,17	17,07	16,87	17,85	19,91
w_n	[%]	51,3	32,5	25,3	27,3	25,1	24
$\square_d$	[g/sm^3]	1,11	1,32	1,39	1,35	1,45	1,64
$\square_d$	[kN/m^3]	10,89	12,96	13,62	13,25	14,27	16,06
n	[-]	0,59	0,52	0,49	0,50	0,46	0,38
e	[-]	1,47	1,07	0,95	1,01	0,86	0,62

w_r	[%]	53,56	39,22	35,11	37,11	31,84	23,49
S_r	[-]	0,96	0,83	0,72	0,74	0,79	1,02
□$_{sr}$	[g/sm^3]	1,68	1,75	1,74	1,72	1,82	2,02
□$_{sr}$	[kN/m^3]	16,48	17,17	17,07	16,87	17,85	19,83
□'	[g/sm^3]	0,68	0,75	0,74	0,72	0,82	1,02
□'	[kN/m^3]	6,67	7,36	7,26	7,06	8,04	10,02

The following values of the settlements were obtained:

- Settlements were observed immediately after explosions.

  $s_{i,rap}=h_i\cdot(\sim 0{,}01\div 0{,}04)$ $\quad\quad$ $s_{rap}=0{,}319$ м
  h_i – the thickness of the layer of weak soil
- The calculation of consolidation after explosions :
  The average module of compression device: $M_0=15144$ МПа
  The average coefficient of permeability: $k=4{,}15\cdot 10^{-9}$ м/с
  Consolidation radius of drains: $r_e=2{,}83$ м
  $r_w=0{,}30$ м
  The geometrical coefficient: $N=r_e/r_w=9{,}43$
  the degree of consolidation: $U=0{,}95$
  The coefficient of horizontal consolidation:
  $$c_R=\frac{M_{0\acute{s}r}\cdot k_H}{\gamma_W}=6{,}28\cdot 10^{-6}\frac{m^2}{s}$$
  The function of the geometry of column-drains:
  $$F(N)=\frac{N^2}{N^2-1}\cdot\ln(N)-\frac{3N^2-1}{4N^2}=1{,}52$$
  The factor of time for horizontal consolidation:
  $$T_R=\ln\left(\frac{1}{1-U}\right)\cdot\frac{F(N)}{2}=2{,}28$$
  The time of horizontal consolidation:
  $$t=\frac{T_R\cdot r_e^2}{c_R}\cdot\frac{1}{86400}=33{,}6$$ days
  Settlements were observed during the process of consolidation, taking into account previous settlements caused by microblasting: $s_1=0{,}034$ m
- Settlements after the construction of embankments:
  The settlements of layers, which are situated below the strengthening zone, are as expected.

The settlements are taken into account to the level when $\sigma_{zd} \leq 0{,}3 * \sigma_{z\square}$.

Table 5- The settlements

The layer	h_i	level/mark	η_S	q_{tot}	σ_{zd}	$0{,}3 * \sigma_{z\square}$	s_2
[-]	[m]	[m]	[-]	[kPa]	[kPa]	[kPa]	[m]
	0,000	0,00	1,000		0,00	0,00	
15ам	0,591	0,59	0,993	150,80	149,75	0,59	0,0008
15атг	0,985	1,58	0,981	150,80	148,00	2,27	0,0008
15атг	1,084	2,66	0,969	150,80	146,09	4,55	0,0008
11атг	0,985	3,64	0,957	150,80	144,37	6,82	0,0002
11атг	0,985	4,63	0,946	150,80	142,65	8,97	0,0002
11атг	0,985	5,61	0,935	150,80	140,94	11,11	0,0002
15атг	0,788	6,40	0,926	150,80	139,58	13,06	0,0006
15атг	0,788	7,19	0,917	150,80	138,23	14,80	0,0006
15атг	0,788	7,98	0,908	150,80	136,89	16,54	0,0006
15атг	0,985	8,96	0,897	150,80	135,22	18,49	0,0007
11атг	0,985	9,95	0,886	150,80	133,56	20,65	0,0002
11атг	0,985	10,93	0,875	150,80	131,92	22,80	0,0002
11атг	0,985	11,92	0,864	150,80	130,29	24,94	0,0002
11атг	1,084	13,00	0,852	150,80	128,51	27,20	0,0002
15ам	0,788	13,79	0,844	150,80	127,23	29,16	0,0009
15ам	0,887	14,68	0,834	150,80	125,80	30,84	0,0010
15ам	0,887	15,56	0,825	150,80	124,38	32,61	0,0010
15ам	0,887	16,45	0,815	150,80	122,98	34,39	0,0010
15ам	0,887	17,34	0,806	150,80	121,58	36,16	0,0010
11ам	0,887	18,22	0,797	150,80	120,20	37,99	0,0003
11ам	0,891	19,11	0,788	150,80	118,83	39,87	0,0006
11ам	0,891	20,00	0,779	150,80	117,47	41,76	0,0006

11ам	0,891	20,90	0,770	150,80	116,12	43,64	0,0006
11ам	0,896	21,79	0,761	150,80	114,78	45,54	0,0006
11ам	0,995	22,79	0,751	150,80	113,31	47,54	0,0006
11ам	0,995	23,78	0,742	150,80	111,86	49,65	0,0006
9бтг	2,000	25,78	0,723	150,80	108,99	53,11	0,0054
9бтг	2,000	27,78	0,704	150,80	106,20	57,94	0,0052
9бтг	2,000	29,78	0,686	150,80	103,48	62,76	0,0051
9бтг	2,000	31,78	0,669	150,80	100,84	67,58	0,0050
9бтг	1,400	33,18	0,657	150,80	99,03	71,68	0,0034
12нв	0,700	33,88	0,651	150,80	98,14	74,42	0,0017
12нв	0,800	34,68	0,644	150,80	97,14	76,68	0,0019
12нв	0,900	35,58	0,637	150,80	96,02	79,23	0,0022
12нв	1,000	36,58	0,629	150,80	94,80	82,09	0,0024
12нв	1,100	37,68	0,620	150,80	93,47	85,25	0,0026
12нв	1,500	39,18	0,608	150,80	91,71	89,15	0,0034
12нв	1,500	40,68	0,597	150,80	89,98	93,66	0,0034
12нв	1,500	42,18	0,585	150,80	88,29	98,17	
12нв	1,500	43,68	0,575	150,80	86,64	102,68	
12нв	1,500	45,18	0,564	150,80	85,03	107,19	
							0,057

The expected settlement during construction of the embankment is: s_2=0,057 m

From the given information it is clear that the microblasting method is more suitable for the soil strengthening of the object for the following reasons:

- most of the settlements occur immediately after the explosion;
- time of the consolidation is 33.6 days;
- the settlements which occur during time of the consolidation and construction of the road embankment have a small value: 0.034 m and 0.057 m

At the same time when drilled piles are used the time of consolidation is 37 days. And after this period the settlements are 0,575 m.

Also it is necessary to take into account the cost of these methods based on this project. From the local estimates (Appendix 4, Appendix 5) we can see that the cost of these works with the help of the microblasting method is nearly twice cheaper.

The results are obtained and presented in Table 5.

Table 5 - comparison of methods

	Microblasting method	Drilled sand piles
by the time of consolidation	most of the settlements occurs immediately after the explosion	37 days
by the cost (thousands, rub.)	88031,71	168126,7

5. Conclusion

There are a lot of different methods of soil improvement. In this work more attention was paid to the microblasting method. This innovative method is very interesting and in our time. It begins to be a serious competitor for alternative methods.

Of course it has some disadvantages:

- impossibility of application in built-up areas;
- ineffectiveness of soil compaction above groundwater level;
- necessity of prior verification of the technology adopted on a test area.

But based on theoretical information and practical example it was noted that soil compaction by the explosion method has many advantages:

- improvement of up to 10 000 000 m^3 of subsoil per month;
- reduction of soil settlement in terms of live load to a level which is significantly lower than specified by both Polish and foreign constructing standards;
- the depth of ground to be improved is theoretically unlimited;
- attaining high values of improved subsoil's strength parameters;
- continuous quality monitoring of subsoil's improvement;
- much faster subsoil consolidation in comparison with other soil improvement technologies;
- low cost of performance in comparison with other soil improvement methods.

Owing to its evident advantages, the explosion method has been widely used in Poland in civil engineering practice during the last 15 years. It should be pointed out that all the described structures built on subsoils improved by the explosion method have shown good stability and no damage during several years of operation.

Since 1991, the technology of improving subsoil using the explosion method has been protected by Polish patent №2370595.

6. References

1) Charlie W.E., Jacobs P.J., Doehring, D.O. (1992): Blast-induced liquefaction of an alluvial sand deposit, Geotechnical Testing Journal 15(1), pp. 14-23.
2) Dembicki E., Imiołek R., Kisielowa N. (1992): Soil compaction with the blasting method, In Geomechanics and Water Engineering in Environmental Management, Chapter 20, edited by R.N. Chowdhury, Balkema, pp. 599-622.
3) Dembicki E., Kisielowa N. (1983): Technology of soil compaction by means of explosion. Proc. 8th European Conference of Soil Mechanics and Foundation Engineering, Helsinki, pp. 229-230
4) Dembicki E., Zadroga B., Bona B., Imiolek R., Kisielowa N., Semrau I. (1987): Changes in groundwater levels due to dynamic compaction of subsoil by hidden underground expolosions, Proc. 9th European Conference of Soil Mechanics and Foundation Engineering, Dublin, pp. 599-602.
5) Ivanov P.L. (1983): Prediction and control techniques to compact loose soils by explosions, Proc. 8th European Conference of Soil Mechanics and Foundation Engineering, Helsinki, pp. 253-254.
6) Dembicki E., Zadroga B.(1997): Polish experience in soil improvement using explosion techniques,Geotechnical Department, Gdansk Technical University, Gdansk, Poland, pp. 19-24.
7) Gohl W.B., Jefferies M. G., Howie J. A., Diggle D.(2000): Geotechnique. Explosive compaction: design, implementation and effectiveness, Canada, pp. 657-665.
8) Mangushev R.A., Karlov V.D., Saharov I.I.(2009) Soil mechanics, Russian, pp. 141-160.
9) State Road Scientific Research Institute, Method of calculation of embankment's settlements(2002), Moscow, http://doc-load.ru/SNiP/Data1/46/46387/index.htm