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Stabilization of Earthwork by Introducing Reinforcing Additives When Mixing Soil

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***Abstract*: In complicated operating conditions of railroads, it is necessary to ensure earthwork stability. The article studies earthwork stabilization methods. The specifics of exploration are represented for each method. In particular, the article considers methods for the stabilization of soil of unstable slopes of embankments on a strong footing and of recesses; embankments on swamps and weak soils; embankments on thawing permafrost soils; stabilization of embankments on thawing permafrost soils; deep reinforcement of soils.**

***Index Terms*: railroads, earthwork, defects and deformations, slipouts and slumps of slopes, weak footing, exploration methods, soil mixing with cement and lime, mixing units.**

I. INTRODUCTION

When railroad operating conditions in Russia change (increased axial loads and loads per unit length of cars, increased movement speed of freight and passenger trains, growing freight traffic density of lines, adopting mechanized methods to handle and repair track infrastructure), it is required to ensure the stability of earthwork that acts as a basis for a railroad track on the most part of the road.

II. METHODS

A. General Discription

The technology of earthwork stabilization by introducing reinforcing additives when mixing soil is intended for regions with seasonal soil freezing and permafrost.
Soil is stabilized in the following conditions:

- soil stabilization on unstable slopes of embankments on a strong footing and of recesses;
- stabilization of embankment footing on swamps;
- stabilization of embankment footing on weak soils;
- stabilization of embankment footing on thawing permafrost soils;
- deep reinforcement of soils.

B. Algorithm

Exploration of soil stabilization on unstable slopes of embankments on a strong footing and of recesses

Places with deformed slopes are identified using the unstable and deforming earthwork passport (PU-9) of track distances. **Slipouts and slumps of embankment slopes** are described by shifted soil masses in the area of slopes, roadsides, sometimes in the under-rail footing due to increased damping of soil in the period of intensive rains in autumn, in abnormally warm winters with liquid precipitation and during soil thawing in spring. Deformations occur due to ill provided moisture drainage in places with ballast recesses on the main field site. Slipouts and slumps are predominantly realized on the side of ballast tanks and beds, sometimes covering the embankment footing. The separation wall goes almost vertically to the depth of 2-4 m (Fig. 1).

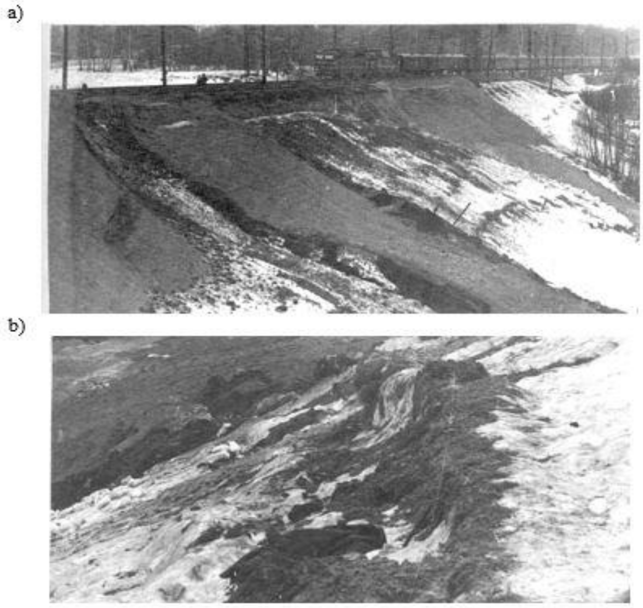

Fig. 1. Embankment slopes' slipout: a) general view; b) frozen layer hummock view on the landslide tip

Deformations are usually expressed on one side of the embankment, the other side remaining stable.During the exploration of embankments with slipouts, pits are made (wells, test pits, stripping) along with exploration channels. Additionally, geophysical methods are used such as ground penetrating radar and the dynamic penetration test (DPT).When exploring deforming areas, a LOZA radar is used [1], a portable pulse radar for subsurface probing of high capacity displaying radar profiles during measurements. As compared to famous foreign and Russian analogs, it has a huge power potential that allows working in environments with high conductivity such as clay loam or damp clay, which is impossible for other radars due to low potential.

DPT that includes current logging and dynamic probing [2] allows dividing soils by their lithological composition (current *I*, mA). The soil type is also determined.

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The condition and physical and mechanical characteristics of soils are determined upon conditional dynamic resistance P_d, MPa. The distribution nature of P_d and I by depth is given in Fig. 2.

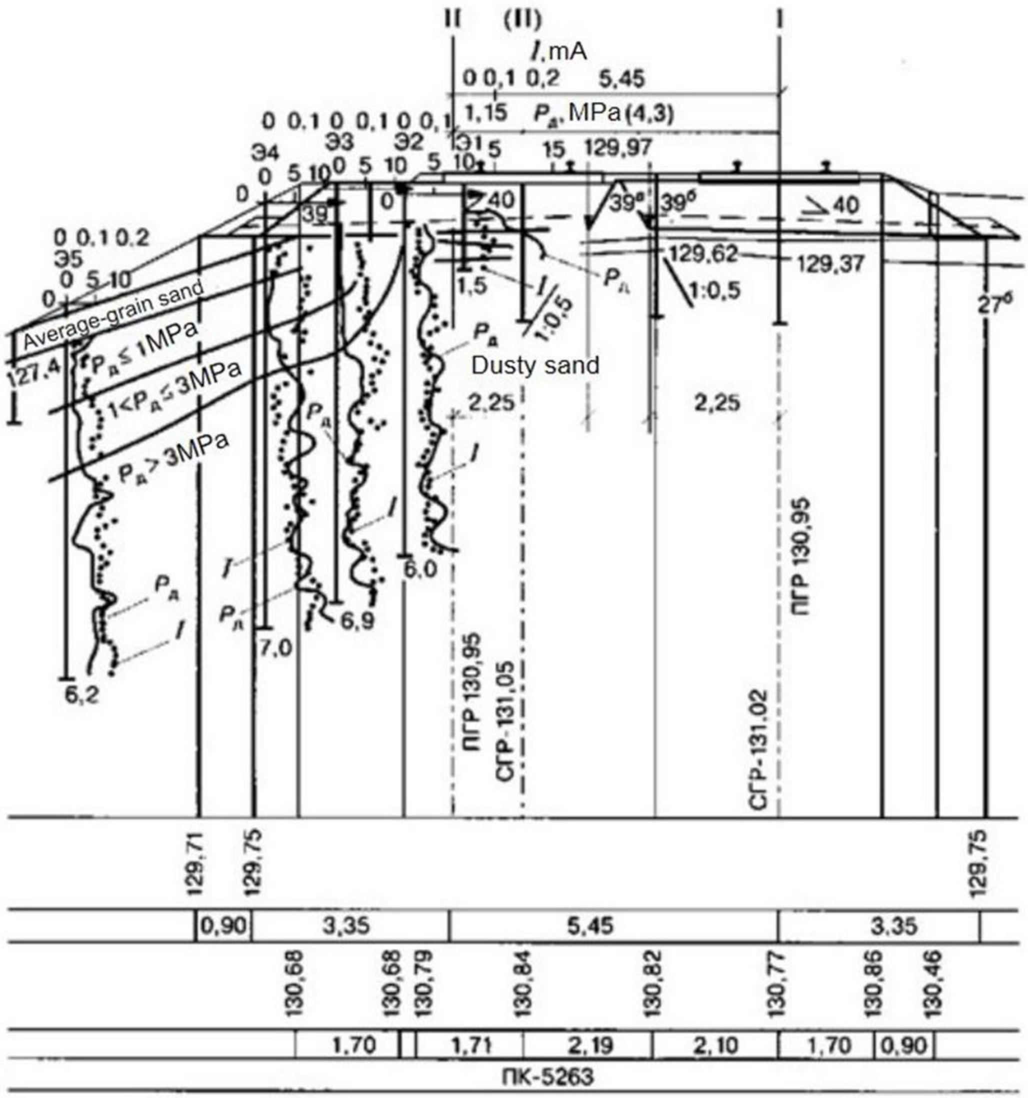

Fig. 2. Cross-profile of earthwork with track II shifting in weak soil condition: Pd – conditional dynamic resistance, MPa; I – current, mA

The measurement range of the parameters Pd and I in soils of different kinds is given in Table 1.

Table 1. Variance range of conditional dynamic resistance and current in soils

Soil		Conditional dynamic resistance P_d, MPa	Current, I, mA
Broken stone ballast		2 to 20	0 to 0.09
Gravel and pebble soil		9 to 23	0.01 to 0.13
Low-moisture sands	large, medium, small	0.5 to 21	0.01 to 0.12
	dusty	1.5 to 12	0.05 to 0.18
Water-flooded sands	large, medium, small	1 to 17	0.05 to 0.20
	dusty	1 to 22	0.1 to 0.24
Sand clay		2 to 17	0.1 to 0.28
Loamy clay		0.3 to 22	0.16 to 0.6
Clay		1 to 21	0.25 to 1.05

Sludge		0.5 to 3	0.28 to 0.85
Decay ooze		0.5 to 11	0.26 to 0.55
Turf	lowland	0.2 to 5	0.1 to 0.5
	highland	0.2 to 2.5	0.02 to 0.1

Recesses are embedded at the site with deformations and beyond it along the entire cross-section of the embankment. Their number is defined by the nature of occurring deformations.

When using DPT, conditional dynamic resistance and current logging are used to determine:

- contours of the interface (contact surface) of draining and ballast materials and clay soils of the embankment within the mine site and slopes;
- condition and properties of these soils;
- dimensions of overmoisturized and weak zones.

Within the main site, DPT is performed in five longitudinal cross-sections:

- along the track axis;
- in subrail zones (on the external side);
- 40 cm from the cross-tie ends.

Based on the data obtained, transverse and longitudinal engineering-geological sections are built. When analyzing longitudinal sections in subrail sections and on roadsides, an opportunity is determined for intrasoil drainage of infiltrating atmospheric precipitation along the ballast bed bottom from highland adjoining sites into the pits (concave elements) of the profile and overdampening of associated soils of the embankment. Places, where the slope has drifted along the side of the ballast recess near cross-tie ends with a breakdown of this side, are identified on cross-sections. As a result of this drifting, the clay soil surface within the field roadside goes down and is buried under draining soils that have been placed when restoring the slope. In these places, the water drainage with the main field site (interfaces of draining accumulated ballast materials and draining materials and clay soils of the earthwork) will be ensured, but water drips and stagnates in dishes formed on the slopes, which causes further losses of slope stability. To identify these dishes on the slopes, probing using a DPT unit in exploration cross-sections must be done at least every meter to get deep into clay soils for 0.1 to 0.3 m. The above exploration transverse slits are not penetrated in this case.

Explorations of unstable embankments on a strong footing must be done directly after the collection of soil thawing before moisture starts to migrate from overmoisturized periphery layers on the main site and in sloping parts deep down the embankment. Slipouts and slumps of slopes in these conditions are caused by overmoisturizing of clay soils by infiltration of atmospheric precipitation and water stagnation in ballast recesses (tanks and beds) on the main site and in dishes on the slopes formed as a result of tipping of individual parts of slopes during slipout.

Slipouts and slumps of recess slopes. Slipouts of recess slopes are characterized by shifting of the soil upper layer 1 to 2 m thick with the overall stability of the slope preserved. Slipouts differ from washouts in a solid shift of surface layers of the soil and differ from slope slumps in a small gripping depth. At an early stage, short cracks occur near the surface and slope edges, and vents near the slope footing, hydrophilous vegetation appears, and slope surface is swollen. For slipouts related to soil freezing and thawing, drips of liquefied mass appear, then soil upper layers 0.3 to 2 m are shifted.

During the exploration of places with slipouts and slumps of slopes, the length of these places is measured along with the propagation along the slope height and the specifics of occurrence as per the above soil characteristics.

Areas with soil protrusion in the recesses are characterized as follows:

- distortion of ditches and other drainage structures in plane and section;
- water stagnation;
- distortion slope contours with breakoff of large masses and occurrence of cirque cracks;
- frequent distorted conditions of the track superstructure in level, plane, and section with the upheaval of rail threads.

The protrusion is caused by weak clay soils at the level of the main site and below it, which are protruded under the weight of soil of high slopes of recesses with the formation of the zone of uneven soil upheaval in the under-ballast footing. Intensive soil settling and swelling in the conditions of regular water-flooding cause plastic deformations.

In places with deformations and beyond them, wells are arranged in recesses for soil sampling and testing and DPT is performed.

Soils of embankments and recesses belong to weak soils with the conditional dynamic coefficient $P_d \leq 3$ MPa.

Exploration of embankments on swamps and weak soils

Penetrations in swamps and weak soils (footings) are provided at least every 200 m in the longitudinal direction.

In the areas of seasonal thawing in dispersive soils using the DPT, the number of wells in each cross-section can be reduced to one, which will act as a reference well. All wells made to 5-15 m can be the reference ones. DPT is performed instead of other wells with the above frequency and depth.

During exploration when crossing a swamp less than 200 m long, penetrations are made near edges and in the middle. Penetrations are made at least 1 m deep from the mineral bottom of the swamp or roofing of strong soils underlying weak soils (sludges, decay oozes, etc.).

Weak soils must be penetrated at least 10 m deep.

On the sections of the road made over the highland swamp on the water parting surface, slope or terrace, cross-sections are located every 200 m.

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On the road crossing a swamp in a bowl-like basin with board slopes steeper than 1:20, cross-sections are located every 50 m. They are used to make an axial well and side wells every 20 m, two or three (in the middle of the swamp) on each side. Additionally, several probing wells are made in a place where the road crosses a pronounced drain stripe along the water course.

When crossing the swamp with a road length below 100 m, the cross-sections are divided every 25-50 m. There must be at least two cross-sections on each swamp.

When crossing bottom land dead lakes or flows, the swamp is probed so that it would be possible to build geological sections on valley lines and cross-sections through an extended swamp.

Wells in each cross-section are embedded along the earthwork axis, near the slope bottom and 25 m from the earthwork axis.

When making wells, peat is sampled layer by layer, but at least every 1 m, to determine the moisture content, the degree of decay, the botanic composition, and ash content. For other weak soils, the organics content, the limits of plasticity, and the moisture content are determined, along with the composition and contents of salts in salty lithologies.

Soils are tested for ultimate shearing resistance determined by a shear meter (spinner) or conditional dynamic resistance is found by DPT.

Swamps are divided into the following types according to the exploration results:

I – peat layer mainly dried, compressed without protrusion under load from the embankment up to 3 m high, having shear resistance in the mass τ above 0.02 MPa or conditional dynamic resistance of $P_d > 2.5$ MPa;

II – layer of peat and other swamp soils protruding under the load from the embankment of 3 m, characterized by $0.003 < \tau \leq 0.02$ MPa or $P_d < 2.5$ MPa;

III – layer of excessively damp soils, including floating bog or sapropelic sediments in the bottom layer, protruding under the load and characterized by $\tau \leq 0.003$ MPa.

The swamp type is determined taking into account the peat compositions and physical properties, Table 2.

Table 2. Swamp type

Name of peat types	Natural humidity, %	Swamp type for turn decay degree, D_{dp}, %		
		Below 20	20 to 45	Above 45
Dried or compacted	Below 300	I	I	I
Low-moisture	300 to 600	I	I-II*	I-II**
Average moisture	600 to 900	I	II	II
High moisture	900 to 1,200	I	II	II
Excessive moisture	Above 1,200	II	II	III
* Type I must include peat with the moisture above 500%. ** Type II must include peat with the moisture above 400%.				

The type and degree of peat decay are determined visually upon the following external signs:

low-decayed ($D_{dp} < 20\%$), light-brown or yellow peat, consisting of nondecayed fibers of mosses and grass plants, light-yellow or yellow-brown water easily squeezed from the elastic mass;medium-decayed ($D_{dp} = 20 \ldots 45\%$) – brown or dark-grey-brown peat with mosses, ling and cotton grass roots, water of opaque brown or coffee color, squeezed in drops, peat slightly stains hands and has prominent elasticity; heavily decayed ($D_{dp} > 45$ %) – dark-brown or earth-black peat with ashy shade, water is not squeezed, and mass protrudes between fingers and leaves heavy stains on hands.Embankment collapses on peat swamps (or sediments) are usually quick (during several hours), at first in the form of uneven settling, then suddenly breaks are formed along the embankment footing with protrusions 5-15 m from the earthwork and quick sagging of embankment parts into breaks of the swamp covering layer. At the stages preceding the fast collapse, isolated sags of swamp surface extended along the embankment and filled with water are formed; upheavals with longitudinal breaks of the floating bog are found on the swamp surface along the slope footing and behind lower areas (towards the field). The road condition in the areas of possible collapses is abruptly aggravated; longitudinal ditches and peat receivers are deformed distorting the longitudinal profile of ditch bottom and shifts of their slopes. Collapses are caused by nondecayed peat layers in the embankment footing that are deformed with protrusion of swamp sediments from under the embankments; noncompacted sludges with the layer thickness of 5-10 m and a relatively strong crust 1.5-2.5 m are typical of lowland areas with dominating lake lagoon sediments and for shores of fresh-water basins. Collapses of these crusts and attenuation of sludges may cause abrupt deformations of embankments. The growth of train loads and vibrations of rolling stock increase the possibility of collapses.

Exploration of embankments on thawing permafrost soils

Exploration in permafrost areas must include a geocryological survey. This survey using space and aerial photographic material determines the areas of thermokarst formations, ice build-ups, upheaval bumps, ice-breaking cracks, thermal erosion, thermal abrasion, solifluction, rock streams, and bogs.

To delineate formations, lenses, veins of underground ice or soil veins in it, subgelisol or permanent snow patches, islands of permafrost soils, thermokarst cavities, horizons of above-, inter- and underfrozen waters, geophysical prospecting is used (ground penetrating radar, electrical and seismic survey).

The position of frozen formations by depth is identified when making wells at least every 200 m. They are positioned 4 m below the thawing boundary.

When the road crosses formations and lenses of underground ices, islands of permafrost soils and subgelisol, exploration cross-sections are assigned every 25-50 m from three wells: along the road axis and on edges of the earthwork design outline.

Penetrations along the road in the areas crossing peat bogs and bogs are made every 100 m. In the case of shorter length of these areas, penetrations are assigned in the beginning, middle and in the end.

Wells at least 3 m deep in the beginning, middle and in the end must be made along the axes of anti-icing soil embankments, icing and frozen belts and water drains.

The time for a geocryological survey, e.g., the period of maximum freezing, April, or the period of maximum thawing, September-October, is assigned on a case-by-case basis depending on the type of frozen formations or processes.

Exploration of settling and creeping embankments on thawing permafrost soils comprises road leveling upon rail heads.

Fist leveling results are used to draw a longitudinal profile of the road for each rail thread, scaled as follows: 1:500 horizontal and 1:5 vertical. After the second and third leveling, road marks are superimposed on the existing profile.

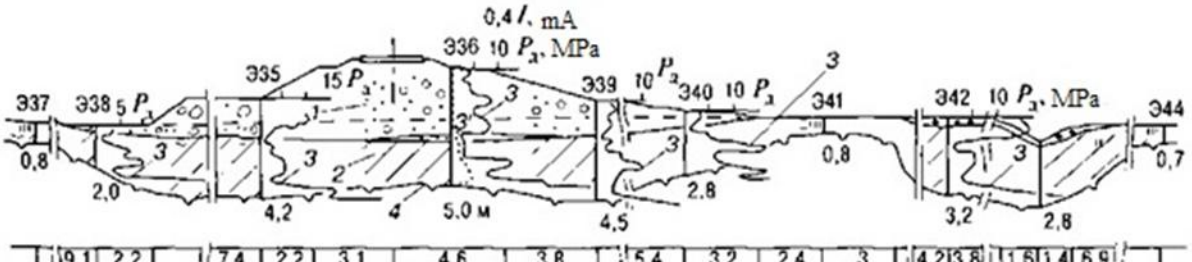
Exploration areas are identified using the passport of road distances and field inspection. Exploration is done for the first time in autumn at the time of maximum thawing (September-October). The road is then leveled during the maximum soil freezing (April) and again in the next autumn. Plane surveying is done simultaneously over fixed points (marks, stakes, stones) in specific transverse gates of the deforming area that must appropriately reflect the extreme (minimal and maximum) values of these settling irrespective of track raising to be done.

Plane surveying measures inclined distances between fixed points on marks with a metallic ruler with the accuracy of ±1 mm calculating horizontal distances using altitude survey results. Combined longitudinal profiles are built upon level results upon rail heads and marks. Wells are drilled in places identified on profiles with maximum and minimal settling, with soil sampling for analysis and frozen soil core analysis during maximum thawing. Wells are equipped for thermometric surveys that are done with the specified periods along with leveling.

Permafrost soils are not divided by their temperature into low temperature and high temperature for the railroad earthwork. These soils are characterized as frozen or thawing.

The full core is sampled during drilling. During frozen soil sampling, its cryogenic structure is identified (massive, layer or netted), as well as the total thickness of ice layers and lenses, and relative settlement during no-load thawing as per the requirements of VSN 61-89 [3].

The survey must be at least one year long. Additionally to well drilling using DPT, the condition, composition, and contents of thawed soils of earthwork and footing are defined, identifying weak zones as shown in the example given in Fig. 3.



1 – draining soil of embankment; 2 – clayey soil of footing; 3 – conditional dynamic resistance Pd; 3' – current I; 4 – upper permafrost boundary

Fig. 3. Engineering-geological section of settling embankment on the bog

Based on the multitude of data obtained, causes for intensive irregular settlements and embankment creeping are defined. This system of exploration is provided for each landscape type (see above) distributing this data to other similar objects where single exploration is done within the required scope: penetrations are made and the DPT is done during maximum soil thawing.

To determine opportunities for water drainage from low areas of thermokarst nature through existing bridges or pipes and to identify the need to arrange additional culverts, an altitude survey is performed within the required scope.

Exploration of karstified and undermined areas

During surveys and deciphering of space and aerial photos, surface karst manifestations are identified. When studying outcrops, petrographic composition, jointing, and rugosity of rocks, surface water flows, and water basins, areas of water absorption and outlets of underground water are reflected.

Geophysical exploration (ground penetrating radar and DPT) identifies the largest karst cavities located closer to the surface, areas of increased jointing, rugosity and water flooding of rocks, as well as tectonic faults and decompressed zones in the covering layer. LOZA radars and DPT are used. Well drilling identifies the roofing thickness of karst cavities revealed within the boundaries of the earthwork, the vertical size of these cavities, the presence, composition, and condition of aggregate in them. Wells go within the cavity. If necessary, a well shifted to the earthwork axis is also embedded. Wells are deepened for at least 5 m below the bottom of the opened cavity.

In the areas coming to undermined areas, changes in geotechnical conditions are identified: collapses, shift molds and settling of the day surface; disturbed drainage of surface water, increased or decreased levels of underground waters are found. Based on the obtained data, further possible changes in these conditions as a result of earthwork operation are predicted.

III. RESULTS AND DISCUSSION

A. Soil stabilization on unstable slopes of embankments on a strong footing and of recesses

Stabilization of embankment slopes on a solid footing and of recesses is provided upon the results of exploration and calculation of the stability coefficient K_s. Areas with $K_s \leq 1.30$ must be stabilized. Slope stabilization structures include reinforced soil masses shaped as a cylinder 1 m or more in diameter spaced as defined by calculation within the unstable part of the slope. These masses are deepened into strong soils of the embankment. Cylindrical masses are oriented along elements perpendicular to the slope surface. They are formed by mixing soil with binders using mechanized means. Cement, lime, ashes and other materials are used as additives. The distance between mass axes in the longitudinal and transverse directions to the road is taken below 4 m.

The diagram of the reinforced slope of the embankment is given in Fig. 4. A similar technical solution in the recess is given in Fig. 5.

Soil is reinforced by mixing soil with binders, either liquid (wet mixing) or powder-like (dry mixing). Deep mixing is usually done as columns, and surface mixing (up to 3 m deep) is done as a mass. Mechanized equipment and the stabilization unit for deep mixing weigh 50-80 tons and are up to 20 m high. The mass mixing unit weighs about 20 tons and is up to 7 m high.

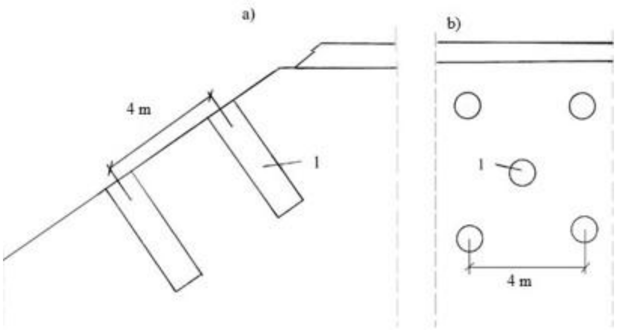


Fig. 4. Stabilization diagram of embankment sloping parts by introducing reinforcing additives: 1 – soil reinforced masses; a) – cross section; b) – longitudinal section

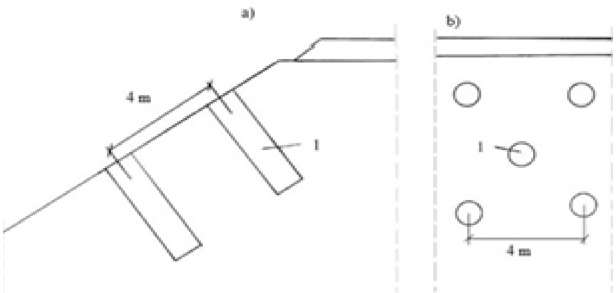


Fig. 5. Stabilization diagram of recess sloping parts by introducing reinforcing additives: 1 – soil reinforced masses; a) – cross section; b) – longitudinal section

Equipment for wet deep mixing of soil (Fig. 6) includes individual vessels for mixing and a feed tank, as well as a pump connected to the deep mixing unit using a flexible pipeline. Mixing is done with high-shear mixers for colloid solutions. Feed tanks are equipped with blade mixers to prevent solution setting-up.The dry method of deep mixing is applied using a standard mixing unit for this mixing (Fig. 7, a) by locating the binder in the hopper of the unit with the air drier and a compressor to get compressed air necessary to supply binder to the mixer. Other designs suggest the location of the hopper, air drier and compressor on individual vehicle chassis that are connected with the mixing unit by a flexible hose (Fig. 7, b).

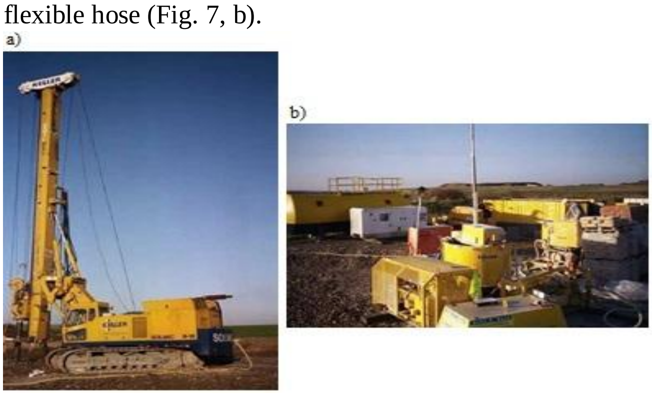


Fig. 6. Deep wet mixing unit a); individual vessels for mixing, feed tank and pump b)

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Fig. 7. Mixing unit for deep dry mixing of soil with the hopper, air drier and compressor a); the same with an individual machine for this equipment b)

Cement is used for the wet mixing method; lime and cement with aggregate are used for the dry method.
Standard mixing devices used for deep dry mixing are given in Fig. 8. They accommodate nozzles to feed binders. The same devices for wet mixing are given in Fig. 9. They have one or several blades and teeth as well as one or more nozzles to feed binders located along the blades.

Fig. 8. Mixing devices for deep dry mixing

Fig. 9. Mixing devices for deep wet mixing
Mixing devices for mixing a dry mass (Fig. 10) can have a diameter of up to 800 mm; they are similar to a ship screw, the binder feeding nozzle is located in the middle.

Fig. 10. Deep mixing device for mixing dry mass

In the case of wet mixing, prepared materials are mixed with water in the mixer with high shear forces until a solution is formed, which contains the specified volume of water and solid particles. The binder solution is then transferred to the vessels where it is constantly mixed to prevent the division of mixture ingredients. The binder solution is pumped from there to the deep stabilization unit at the defined flow rate.
Dry mixing employs dry materials and dried compressed air; the binder and air mixture is then purged directly to the stabilization unit mixing device.
The mathematic model and stability calculation methodology for sloping parts of embankments and recesses suggests as follows.
Embankment stability is determined by calculation with respect to the following specifics of deforming zone formation:

- cracks near field ends of cross ties and slope slipout almost with a vertical breakoff wall up to 2-4 m high on the side of the ballast bed;
- involvement of soils under the assembled rails and sleepers into slipout (sometimes capturing intertrack space in double-tracked segments);
- stability losses of the ballast tail.

The shift of the unstable part of the embankment is provided in design diagrams for the stability test, respectively:

- over the broken sliding surface;
- over the round cylindrical sliding surface;
- over ballast tail bottom.

In the roadside area, soils are loose, and clayey soils, especially at the boundary with draining soils, are in fluid or fluid-plastic condition. The moving load is taken by the prism of stronger soil with steep walls directly under the assembled rails and sleepers. The possibility of stability loss by slope parts is increased when soil is overmoisturized in spring during thawing and in autumn during raining (Fig. 11). In the latter case and in no-frost winters, in climate abnormalities, moisture in the soil moves to the cold surface impeding infiltration of liquid precipitation and overmoisturizing the soil.

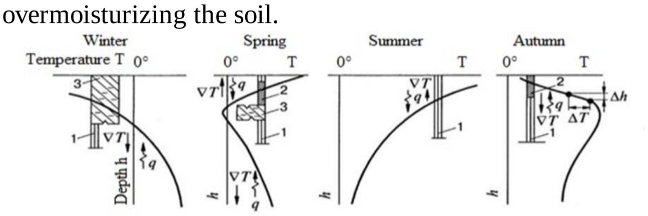

Fig. 11. Diagram of soil temperature distribution by depth and direction of temperature gradients (∇T) and moisture flows (q) by seasons: $\nabla T = \Delta T / \Delta h$; 1, 2 – low-moisturized and overmoisturized soils, respectively; 3 – frozen soils

For a potentially unstable mass divided into blocks (Fig. 12), the stability coefficient is determined using the formula

$$K = \sum_{i=1}^{i=n} \frac{(tg\varphi_i N_i + c_i l_i + T_{i-sp} - U_i)\cos\varphi_i}{\cos(\alpha_i - \varphi_i)} : \sum_{i=1}^{i=n} \frac{T_{i-sh}\cos\varphi_i}{\cos(\alpha_i - \varphi_i)}, \quad (1)$$

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where φ_i is the soil internal friction angle; N_i is the rated block gravity component normal to the possible shift line; c_i is specific adherence; l_i is the rated block length; $T_{i\text{-sp}}$, $T_{i\text{-sh}}$ is the tangential retaining and shearing components of rated block forces, respectively; U_i is the porous pressure; $\alpha_i = \beta_i - \eta_o$; β_i is the inclination angle of rated block possible shift; $\eta_o = \beta_{md} - \varphi_{md}$ is the inclination angle of interaction between blocks for middle or two middle blocks.

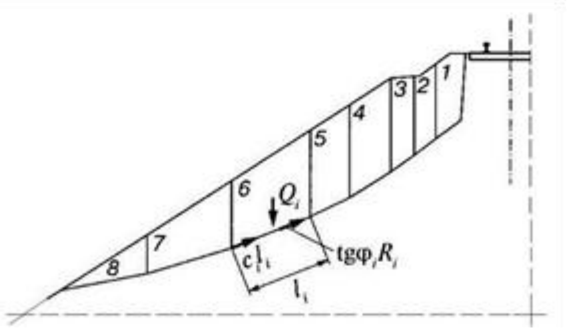

Fig. 12. Calculation diagram for the stability of embankment sloping part: Ri is the footing reaction

Porous pressure is taken into account in the calculation of ballast tail stability (calculation diagram of shift along the bottom) and for embankments composed of sands. It is taken equal to 0.4 t/m^2 for regions of overmoisturizing and to 0.2 t/m^2 for other regions.

For the limit equilibrium condition (K = 1.0) in formula (1), the values of c_i and φ_i are set with respect to the requirements of TsPI-36 [2]. With the parameters found in this formula, the necessary specific resistance $\sum_{i=1}^{i=n} T_{i-\mathrm{sp}}$ is determined for one long meter of slope length. Based on the structural cross dimensions (diameter) of the cylindrical mass (column) taken as b_c= 1 m and the strength characteristics of reinforced soil, the number and location of these masses on deforming slopes of the embankment are calculated and assigned. They are deepened below the potential sliding surface. The depth is determined by calculation. The calculation is done for landslide pressure E, ts/m, from blocks between the rows, see Figs. 4, 5, as per the calculation diagram in Fig. 13.

$$E = \sum_{i=1}^{i=n} \frac{(KQ_i \sin\alpha_i - Q_i \cos\alpha_i \mathrm{tg}\varphi_i - c_i l_i + U_i l_i)\cos\varphi_i}{\cos(\alpha_i' - \varphi_i)}, (2)$$

where K is the slope stability coefficient taken equal to 1.3; Q_i is the weight of each of n blocks that the slope is conditionally divided into above the retaining element; α'_i is the inclination angle of the i-th block bottom, deg.

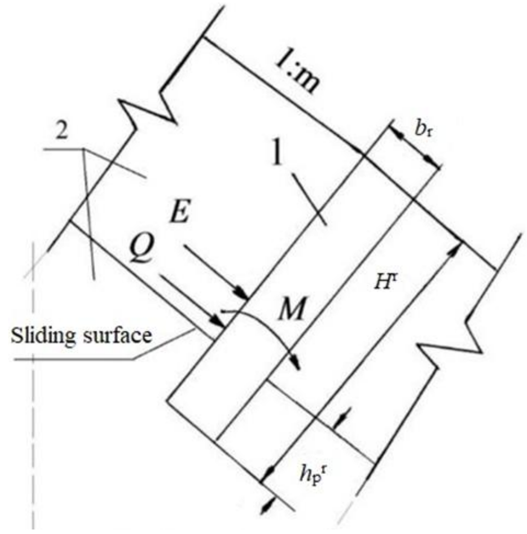

Fig. 13. Calculation diagram for the retaining structure made of reinforced soil mass (1) in the earthwork sloping part (2): E is the landslide pressure; Q is the transverse force; Q is the transverse force; $Q = E$; M is the bending moment from E; b_r is the transverse size of the mass; H^r is the longitudinal size of the mass; h_p^r is the depth of penetration of the mass; 1:m is the slope steepness

The depth of penetration h_p^r is determined using the formula

$$h_p^r = \frac{5Q + \sqrt{25Q^2 + 36M b_r R}}{n b_r R}, (3)$$

where $Q = E$ and M are the rated values of transverse force, ts, and the bending moment, ts·m, respectively; b_r is the transverse mass size equal to 1.0 m; n is the coefficient taken as 2.2; R is the rated stress in soil, ts/m^2.

$$R = \frac{4}{\cos\varphi}(\gamma Z \,\mathrm{tg}\,\varphi - \xi c), (4)$$

where γ is the soil density, t/m^3; Z is the depth where R is measured, which is counted from the soil surface; ξ is the coefficient taken as 0.3.

R is recommended to be taken at the depth of 1.5 m from the rated sliding surface.

For the rated length of the deforming section equal to 100 m, the total shear strength of cylindrical masses must be equal or exceed tangential retaining forces

$$n\frac{\pi b_r^2}{4} R_r \geq 100 \sum_{i=1}^{i=n} T_{i-sp}, (5)$$

where n is the number of masses, units; R_r is the strength of reinforced soil, ts/m^2.

Works to prevent slipouts and slumps of embankment slopes and recesses are divided into preparatory and primary.

Preparatory works include the construction of the passage along the object to be stabilized and field tests to confirm possible compliance with design requirements.

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The passage is to be constructed on weak soils characterized by the conditional dynamic resistance of $P_d \leq 3$ MPa. To do it, stripes of nonwoven material weighing 500 g/m^2 [4] are spread and a layer of gravel (land waste) 0.4 m thick is filled under GOST 25100-2011 [5].

During field tests, the same equipment, materials, technologies, and procedures are used as envisage for primary works. Test operating parameters include as follows:

- mixing tool immersion and extraction speed;
- rotational velocity of the mixing unit rotary device(s);
- air pressure (in the case of dry mixing);
- binder (cement mortar) feed rate.

Primary works are done using equipment and technologies as given below.

Deep mixing permits improving soil characteristics, for example, increasing shear and/or compression strength, by mixing soil with additives that react with the soil substance. This improvement becomes possible due to ion exchange on the clay surface, binding of clay particles and/or pore filling in the soil with the products of the chemical reaction. Types of deep mixing are divided by the type of the applied binder (cement, lime, and cement mixture with possible additives, such as gypsum, ash dust, etc.) and by the mixing method (dry/wet, rotational/jet, drilling or blade).

Works include equipment installation, immersion, and extraction. When the mixer is immersed, it cuts and crushes soil to the depth required for reinforcement. During extraction, the binder is fed to the soil at a constant rate; the mixing tool extraction rate also remains the same. The mixer blades are rotated in the horizontal plane and the soil is mixed with the binder.

Samples are taken in the reinforced soil before this soil starts to set. Samples are taken as follows: the tool is immersed to take a wet sample to the explored depth, a liquid sample is taken, the tool is closed to take a wet sample and the sample is raised to the surface where the material is processed and placed to cylinders for testing. Samples are solidified at the required temperature in molds of standard size. Sample tests are done after 28 days of solidifying.

Samples are taken as cores in the reinforced mass of soil in 28 days of solidifying. Soil testing by uniaxial compression is done to find the ultimate uniaxial compression strength R_c for semi-rock soils as per GOST 12248 – 2010 [6].

The ultimate uniaxial compression strength R_c (MPa) of semi-rock soil is calculated with the accuracy up to 0.1 MPa

$$R_c = 10\frac{F}{A_0}$$

where F is the load when the collapse occurs (kN); A_0 is the initial cross-section area of a soil sample, cm^2.

B. Stabilization of embankment footing on swamps and weak soils

In the areas with embankment deformations on weak soils, mass stabilization using a dry method is applied.

Mass mixing is done using a shovel crawler equipped with the equipment shown in Fig. 14. The binder is cement fed from an individual unit where a hopper, an air drier, and equipment to control material feeding are installed.

a)

b)

Fig. 14. Type of equipment for mixing soil by the mass method (a); the mixing hose and mixing unit on the ground surface (b)

Calculations are done in a differentiated manner for swamps, weak soils and thawing permafrost soils with respect to primary embankment parameters.

The diagram of embankment on a swamp with the calculation diagram to determine the stability coefficient K_s^{sw} is given in Fig. 15. The embankment is composed of draining soils. The stability coefficient for these conditions is measured using the formula

$$K_s^{sw} = \sum_{i=1}^{i=n} \frac{\left(\mathrm{tg}\,\varphi_i Q_i^{sw}\cos\beta_i + c_i l_i + T_{i-sp} - U_i\right)\cos\varphi_i}{\cos(\alpha_i - \varphi_i)} : \sum_{i=1}^{i=n} \frac{Q_i^{sw}\sin\beta_i\cos\varphi_i}{\cos(\alpha_i - \varphi_i)}, \quad (6)$$

where Q_i^{sw} is the gravity of the i-th block in the diagram of embankment on a swamp (see Fig. 15).

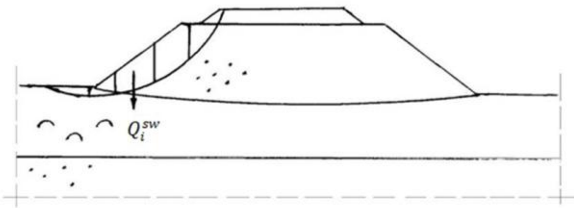

Fig. 15. Cross-section profile embankment on a swamp with the calculation diagram to determine the stability coefficient Kssw

The diagram of embankment on weak soils where embankment and footing are composed of clay varieties together with the calculation diagram is shown in Fig. 16.

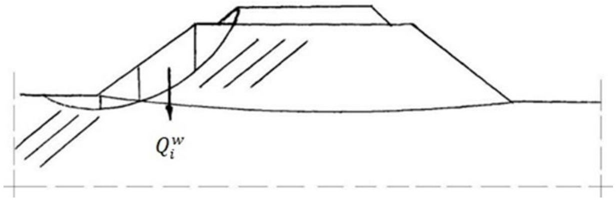

Fig. 16. Cross-section profile embankment on weak soils with the calculation diagram to determine stability coefficient Ksw

The stability coefficient K_s^w is measured using the formula

$$K_w^s = \sum_{i=1}^{i=n} \frac{\left(\operatorname{tg}\varphi_i\, Q_i^w \cos\beta_i + c_i l_i + T_{i-sp}\right)\cos\varphi_i}{\cos(\alpha_i - \varphi_i)} : \sum_{i=1}^{i=n} \frac{Q_i^w \sin\beta_i \cos\varphi_i}{\cos(\alpha_i - \varphi_i)}, (7)$$

where Q_i^w is the gravity of the *i*-th block in the diagram for weak soils (see Fig. 8.3).

To prevent and eliminate settlements and creeping of embankments on swamps and weak soils, it is required to mix peat and these weak soils with additives in the area of the embankment slope footing (Figs. 17, 18).

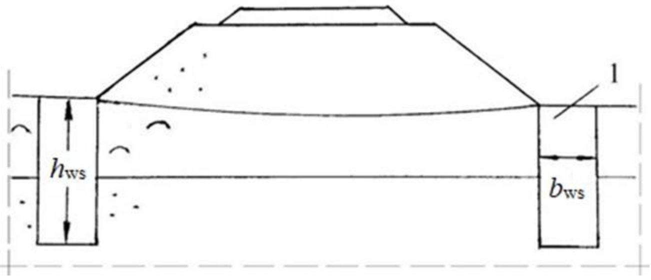

Fig. 17. Diagram of reinforcing embankment footing on the swamp: 1 – reinforced soil; b_{ws} is the width of the wall in the soil; h_{ws} is the reinforcement depth

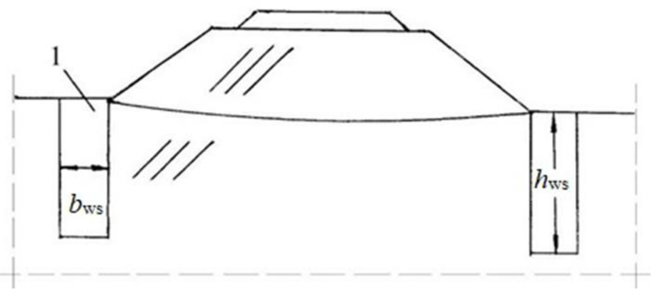

Fig. 18. Diagram of reinforcing embankment footing on weak soils: 1 – reinforced soil; b_{ws} is the width of the wall in the soil; h_{ws} is the reinforcement depth

The reinforcement layer width is calculated as follows. The parameters c_i and φ_i are determined in formulas (6) and (7) for the limit equilibrium condition ($K_s^{sw} = 1.0$; $K_s^w = 1.0$). For the measured parameters, the necessary conditional value of $\sum_{i=1}^{i=n} T_{i-sp}$ is determined to ensure the stability coefficient of $K_s^{sw} = 1.3$; $K_s^w = 1.3$. The strength of the wall in soil must be equal or exceed the sum of tangential retaining forces for 1 long meter of the structure along the way

$$1 \cdot b_{ws} \cdot R_r \geq \sum_{i=1}^{i=n} T_{i-sp}, (8)$$

where b_{ws} is the thickness of the wall in the soil (m); R_r is the density of reinforced soil (ts//m^2).

The diagram of mass stabilization of embankment footing on swamps and weak soils is given in Fig. 19. The mixer combined with an excavator rotates and simultaneously moves in horizontal and vertical planes.

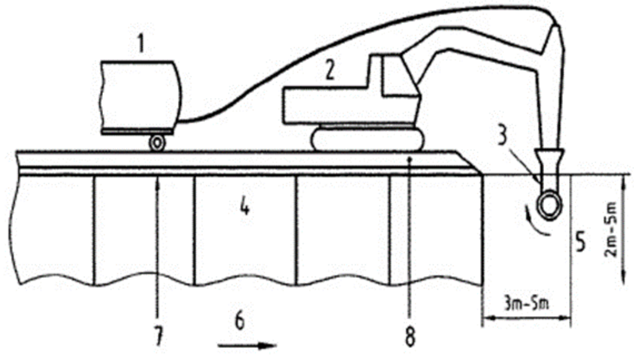

Fig. 19. Diagram of mass stabilization on swamps and weak soils: 1 – tank with a stabilizer; 2 – working machine; 3 – mixer; 4 – treated weak soil; 5 – peat or weak soil; 6 – direction of mass stabilization; 7 – unwoven material; 8 – backfilled gravel

C. Stabilization of embankment footing on thawing permafrost soils

In the areas with embankment deformations on thawing permafrost soils, stabilization is provided in case of settlement intensity over rail heads above 20 mm per year.

The diagram of the embankment of thawing permafrost soils combined with the calculation diagram is given in Fig. 20. The footing is represented by clay soils. The embankment is composed of draining soils. The stability coefficient K_s^{th} is calculated using the formula

$$K_s^{th} = \sum_{i=1}^{i=n} \frac{\left(\operatorname{tg}\varphi_i\, Q_i^{th} \cos\beta_i + c_i l_i + T_{i-sp} - U_i\right)\cos\varphi_i}{\cos(\alpha_i - \varphi_i)} : \sum_{i=1}^{i=n} \frac{Q_i^{th} \sin\beta_i \cos\varphi_i}{\cos(\alpha_i - \varphi_i)}, (9)$$

where Q_i^{tho} is the gravity of the *i*-th block in the diagram in Fig. 20.

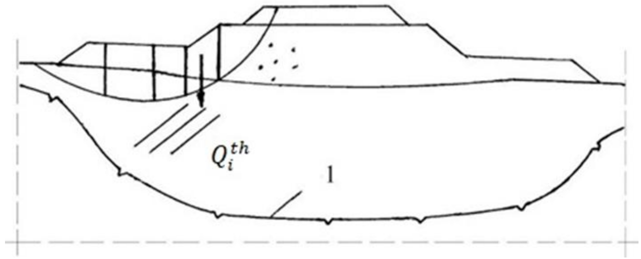

Fig. 20. Cross-section profile embankment on thawing permafrost soils with the calculation diagram to determine the stability coefficient K_s^{th} : 1 – permafrost table (PFT)

When stabilizing embankments on thawing permafrost soils, berm and footing soil are mixed with additives below weak soils (Fig. 21). A wall in the soil is created as a result of these operations.

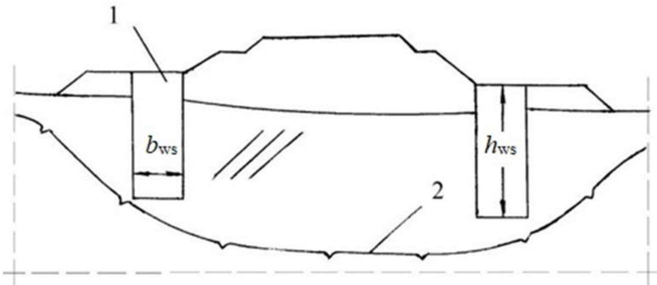

Fig. 21. Diagram of reinforcing embankment footing on thawing permafrost soils: 1 – reinforced soil; bws is the width of the wall in the soil; hws is the reinforcement depth; 2 – permafrost table (PFT)

The diagram of mass stabilization on thawing permafrost soils is given in Fig. 22.

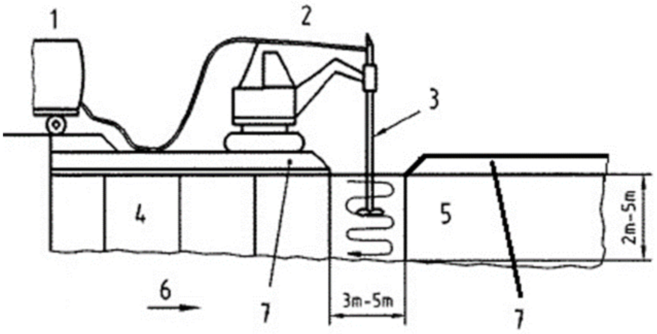

Fig. 22. Diagram of mass stabilization of embankment footing on thawing permafrost soils: 1 – tank with a stabilizer; 2 – working machine; 3 – mixer; 4 – treated soil; 5 – thawing permafrost soil; 6 – direction of mass stabilization; 7 – berm

D. Deep reinforcement of soils

Deep reinforcement of soils is done as columns. Depending on the equipment type, the column diameter is 0.5-0.8 m to 1.0-1.5 m. The columns can be 30-40 m deep when using the largest units.

In the case of deep stabilization with the wet and dry process of mixing, the binder is injected to the soil through a hollow pipe to the mixing device nozzle. In the case of dry mixing, the binder is fed to the mixing device only when it is extracted from the defined mixing depth, whereas in the case of wet mixing, the binding solution is fed during penetration and extraction from the defined mixing depth. Due to the rotation of the mixing device and injection of the binder through the nozzles, the soil mixes with the binder and, as the pipe is extracted, the stabilization column is formed.

The deep reinforcement method is suggested in the following conditions:

- within karst-hazardous areas;
- in places of settling under mines;
- in areas with soil protrusion in recesses;
- in places of long-term settling in approaches to bridges and culverts;
- in case of embankment collapses on peat swamps;
- on landslides.

Karst-hazardous areas. Columns in karst-hazardous areas are arranged within a safety margin of buildings (GOST 9238-2013) on both sides of the earthwork at least 3.1 m from the road axis to the closest edge of the column. Columns can be located within culverts. Columns are located 4.0 m from each other in 1-3 rows along the road. The number of rows, as well as the depth of soil stabilization by columns, is assigned based on exploration data by the geophysical methods of ground penetrating radar and DPT. The diagram of arranging columns of reinforced soil in the karst-hazardous areas is given in Fig. 23.

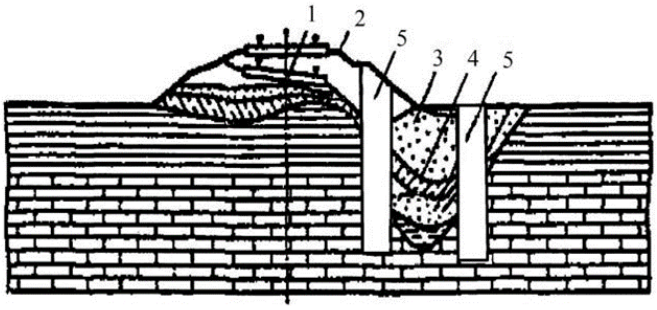

Fig. 23. The diagram of earthwork stabilization in karst-hazardous areas: 1, 2 – rod positions after collapse and restoration, respectively; 3 – soils of the slid part of the embankment; 4 – karst funnel; 5 – columns of reinforced soil

Deep dry mixing is provided for clay soils having the average natural humidity by depth $w_n > 0.2$. The equipment shown in Fig. 7 is used. Dry mixing works are done as per the diagram in Fig. 24. These works include the following operations:

- installation of machines to a working position;
- immersion of a mixing shaft to the required depth with simultaneous soil loosening;
- gradual lifting of the working organ with simultaneous injection of binder to the soil;
- soil mixing with the binder during mixer rotation.

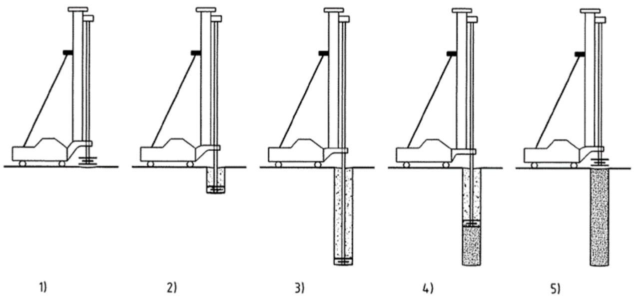

Fig. 24. Diagram for dry mixing works: 1) mixer installation; 2) – 3) mixer immersion; 4) – 5) mixing with binder

Deep wet mixing is provided for clay soils having the average natural humidity by depth $w_n \leq 0.2$. The equipment shown in Fig. 6 is used.Deep reinforcement with columns is provided in disperse soils. Reinforcement is not provided in karstifiable rocks.

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Areas with the settling of earthwork above mines. To stabilize the footing, columns of maximum diameter as indicated above are arranged. The diagram of column arrangement is given in Fig. 25. Columns are arranged at least 3.1 m from the road axis. They are constructed at least every 3.0 m. Depending on the location of the collapse funnel and deformation intensity, columns are provided on one or two sides of the road. Moreover, along the funnel edge that is closest to the road, a number of columns are provided every 5.0 m. Units and type of deep mixing (dry or wet) are adopted depending on the average moisture of clay soil as per the requirements given for karst-hazardous areas.

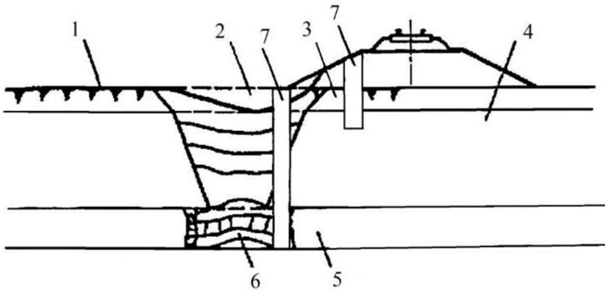

Fig. 25. Diagram of earthwork stabilization on areas with settling above mines: 1 – day surface; 2 – collapse funnel; 3 – disperse soil; 4 – primary rocks; 5 – minerals; 6 – mined-out space; 7 – columns of reinforced soil

Areas with soil protrusion in recesses. Soil reinforcement in recesses by arranging columns is provided along the entire cross-section of the earthwork within roadsides on main areas, ditches, slopes and, if necessary, parts behind slopes. The column row closest to the road is arranged at least 3.1 m from the axis. The distance between columns in the direction longitudinal to the road is taken as 3.0 m. The reinforcement diagram of protruding soils in the recess is shown in Fig. 26. Columns are arranged within the layer of weak soils with embedding into firm soils by at least 4.0 m.

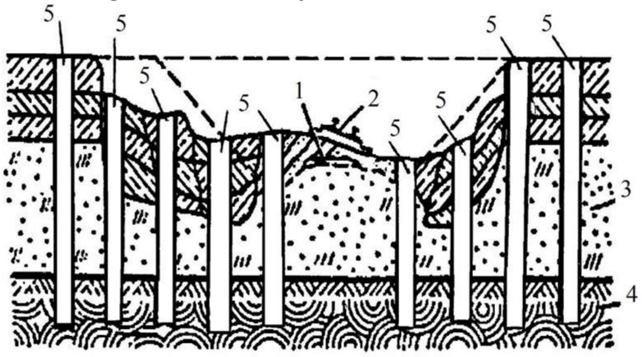

Fig. 26. Diagram of earthwork stabilization in areas with soil protrusions in the recess: 1 – construction position of the road; 2 – road position after protrusion of main site soils and settling of recess slopes; 3 – a layer of weak soil; 4 – firm soil; 5 – columns of reinforced soil

Places of long-term settling in approaches to bridges and culverts. To stabilize deforming areas, columns of reinforced soil are provided. Columns are located in the longitudinal direction directly beyond the bridge piers and culverts on both sides of the road within the area with embankment settling, the length of which is taken according to the exploration date, but no less than 30.0 m. The frequency of column arrangement must be 3.0 m. In the cross-section, the columns must be arranged 3.1 m from the road axis as per the diagram in Fig. 27. The columns are constructed in the embankment and its footing 3.0 m below the layer of weak soil deposits characterized by the conditional dynamic resistance of $P_d < 3.0$ MPa.

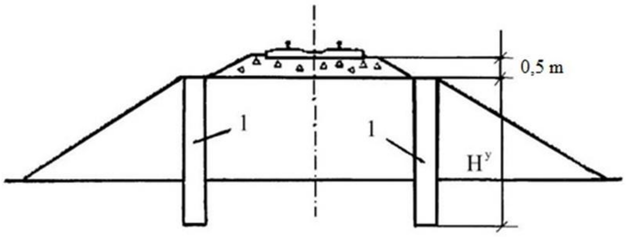

Fig. 27. Diagram of earthwork stabilization at the bridge approach: 1 – columns of reinforced soil

Areas with embankment settling on peat swamps (clay deposits). To stabilize the footing of embankments, column rows of reinforced soil are designed 3.1 m from the road axis on both sides (Fig. 28). Additionally, columns are designed within collapses outline during exploration using the geophysical methods of ground penetrating radar and DPT. The columns must be located every 3.0 m in the longitudinal and transverse directions to the road embedding them for at least 3.0 m into dense (firm) soils characterized by the conditional dynamic resistance of $P_d > 3.0$ MPa. Columns are built on the length of 30.0 m on each side beyond collapses in the longitudinal direction. When collapses are located under the main site, it is necessary to widen it by backfilling the draining soil to the slopes to ensure road shift alternately to both sides.

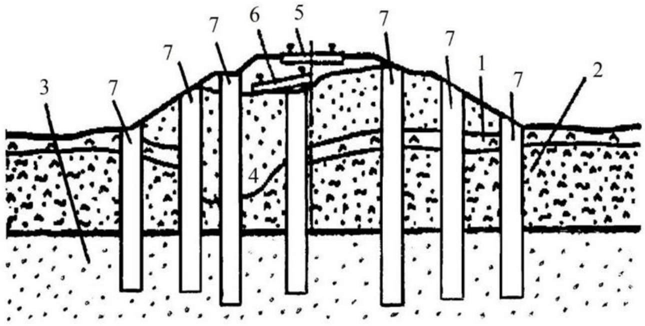

Fig. 28. Diagram of embankment stabilization on peat swamps (sludge deposits) with soil reinforcement: 1 – peat crust; 2 – sludges; 3 – dense soils; 4 – collapse with a breakdown of peat crust; 5, 6 – road before and after the collapse, respectively; 7 – reinforced soil columns

Areas with landslides. To prevent shifts of landslide masses in areas with landslides, columns of reinforced soil are designed under diagram in Fig. 29. Stabilization is provided on the landslide area and behind it along the entire perimeter at least 25.0 m apart. The distance between columns must be taken at least 4.0 m in the longitudinal and transverse directions to the road.

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Columns must be located along the entire layer of the landslide mass and at least 6.0 m below the landslide sliding surface. The column arrangement rate within the earthwork must be at least 3.0 m. Together with the columns, existing devices for drainage of surface and soil water must be restored at the landslide area, swampiness and other water stagnations must be eliminated, and cracks must be patched on the landslide surface.

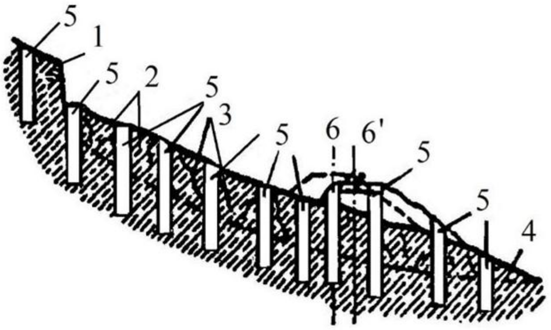

Fig. 29. Diagram of embankment stabilization by soil reinforcement: 1 – break edge; 2 – landslide mass; 3 – cracks in the landslide; 4 – landslide sliding surface; 5 – reinforced soil columns; 6 – road axis before shift; 6' – the same after shifting

Technical solutions are structures and technologies for soil reinforcement by binders developed with respect to national and foreign experience [7-12].

IV. CONCLUSION

Methods of earthwork stabilization by introducing reinforcing additives when mixing soil are given. Stabilization is provided in areas with deformations of embankment slopes on a firm footing and of recesses, embankment footings on swamps and weak soils and on thawing permafrost soils. Deep soil reinforcement by columns is provided within karst-hazardous areas, in places of settling above mines, in areas with soil protrusion in recesses and in areas of long-term embankment settling in approaches to bridges and culverts. Stabilization is done in a mechanized way. The characteristics of the units for deep wet and dry mixing are given. Cement, lime, and other materials are used as binders. Stabilized areas are explored using the geophysical methods of ground penetrating radar and DPT in a differentiate manner by types of deformations and earthwork defects. Methods of calculation of stabilization structures are given.

REFERENCES

1. VNIISMI Company LLC. Geofizicheskii kompleks dlya opredeleniya granits geologicheskikh sloev serii "LOZA": sertifikat sootvetstviya No. ROSS RU.AG93.N01175 [LOZA Geophysical System for Determining Boundaries of Geological Layers: Compliance Certificate No. ROSS RU.AG93.H01175]. 2012.
2. TsPI-36 Rukovodstvo po opredeleniyu fiziko-mekhanicheskikh kharakteristik ballastnykh materialov i gruntov zemlyanogo polotna [TsPI-36 Manual for Determining Physical and Mechanical Characteristics of Ballast Materials and Earthwork Soils]. Approved by the RZD OJSC Road and Construction Department, 2004, January 30.
3. VSN 61-89. Izyskaniya, proektirovanie i stroitelstvo zheleznykh dorog v raionakh vechnoi merzloty VSN 61-89 [Surveys, Engineering and Construction of Railroads in Permafrost Areas]. 1989.
4. TsPI-32 Tekhnicheskie ukazaniya po stabilizatsii zemlyanogo polotna i ballastnogo sloya (dlya opytnogo primeneniya) [TsPI-32. Technical Guidelines to Stabilize Earthwork and Ballast Layer (for Experimental Use)]. Approved by the Road and Construction Department of the Traffic Ministry, 2003, September 18.
5. GOST 25100-2011 Grunty. Klassifikatsiya [GOST 25100-2011 Soils. Classification]. 2011.
6. GOST 12248-2010 Grunty. Metody laboratornogo opredeleniya kharakteristik prochnosti i deformiruemosti [GOST 12248-2010 Soils. Methods for Laboratory Determination of Strength and Deformability Characteristics]. 2010.
7. V.M. Bezruk, "Ukreplenie gruntov" [Soil Reinforcement]. Moscow: Transport, 1995, pp. 340.
8. T.W. Lambe, "Improvement of Strength of Soil-Cement with Additives", *Highway Research Board Bulletin*, 183, 1958.
9. EuroSoilStab, Design Guide Soft Soil Stabilisation. Development of Design and Construction Methods to Stabilise Soft Organic Soils. CT97-0351. 2002.
10. N. Puumalainen, H. Halkola, K. Rantala, J. Forsman, P. Hautalahti, "Mass Stabilisation and Combined Mass and Column Stabilisation in Kivikko Area, Helsinki", In NGM 2004, Ystad, Sweden, May 19-21, 2004.
11. M. Burgos, F. Samper, "Geotechnical Characteristics of a Very Soft Dredged Silty Clay and a Soil-Cement Mix in Valencia Port (Spain)", In Fourth International Conference on Soft Soil Engineering. Vancouver, British Columbia, Canada, Vol. 1, 2006, pp. 427-435. Taylor & Francis Group plc.
12. J. Laugersen, "Stabilisation of Contaminated Sediments in Trondheim Harbour – Special Challenges", In Proceedings of the International Conference on Deep Mixing – Best Practice and Recent Advances, Deep Mixing'05. Stockholm, Sweden, May 23-25, 2005.
13. C. Ladd, "Stability Evaluation during Staged Construction", *Journal of Geotechnical Engineering*, 117(4), 1991, pp. 540-615.