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# Local stability of soil-cement columns above undergraoud excavations

Abstract. The paper studies local stability of soil-cement elements (SC) formed by a jet grouting technology within the zone influenced by underground excavation. The research topicality is stipulated by the accident taken place in 2021 during metro construction in the city of Dnipro and the need to assess risks during excavation in strengthened foundations. The aim of the study is to determine the influence of the geometric parameters of SCs and reduction of their strength on the stability of rock mass above the excavation. The conditions of the accident development are analyzed, including hydrogeological regime, dynamic loads from drilling-and-blasting operations, and disturbance of soil structure. It is defined that deviations of the SC dimensions from the design values and a decreasing strength along the SC lateral surface significantly reduce the excavation roof stability. A considerable influence of dynamic loads on stability loss is confirmed. Practical recommendations are formulated to improve the reliability of geotechnical systems and reduce accident risks.

Keywords: soil-cement columns, jet grouting, underground excavation, dynamic loading, hydrogeological conditions, geotechnical risk

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## Introduction.

The study was motivated by the accident that occurred in 2021 in the centre of the city of Dnipro during metro construction [1] as well as by the earlier related cases [2, 3] and the development of this accident at present [3, 4].

The purpose of this study was to assess the risks that may arise while driving underground excavations in the foundations strengthened with the help of jet grouting [5, 6] and develop recommendations to avoid such accidents in future.

To achieve this goal, following tasks were addressed:

1. To determine how the stability and strength of the soil above excavations change with the varying design dimensions of soil-cement elements (SC).
2. To define how strength reductions along the lateral SC surface (a phenomenon occurring under dynamic loads on the foundation) affect the strength and stability of the rock in the underground excavation roof.

## Main material and results.

According to the data presented in [1], the accident occurred in terms of following conditions:

1. It took place in an inclined shaft constructed in a foundation strengthened by jet grouting. The shaft opening (i.e. a junction of the inclined shaft with the station structure) is located at a depth of approximately 58 m below the daylight surface.
2. When accident happened, the height of a rock column above the face of the inclined shaft was 9-10 m.

3. According to the assumption by the authors of [1], the accident could be probably caused by drilling-and-blasting operations carried out in the face and, as a result, by a reduced strength of the rock (i.e. its decreased angle of internal friction) due to the impact of additional dynamic load.

4. During the site investigations, a stabilized groundwater level was found at an elevation of -21 m relative to the daylight surface; and after the accident it rose up to an elevation of 10-12 m below the daylight surface. At the same time, within the aeration zone, the perched water table was recorded at a depth of 1-1.5 m from the daylight surface.

5. Prior to the collapse of the excavation roof, there was a water seepage from the soil into the tunnel from its crown, followed by the collapse of rock fragments, brick, and concrete into the tunnel.

6. The design length of SC within the accident site is approximately 5 m (Fig. 1).

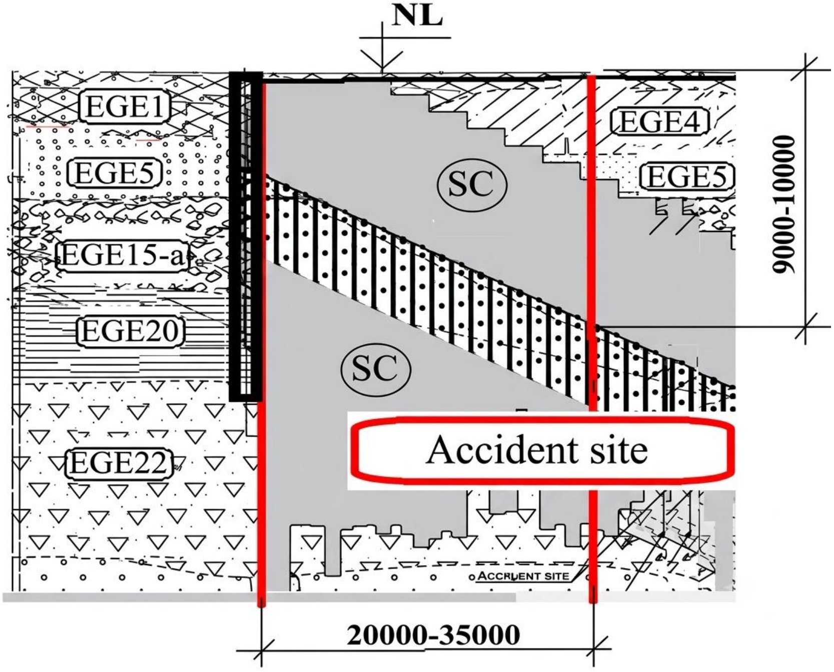

**Figure 1 – Structure of the soil mass.**
**Notes: 1. EGE 1, EGE 4-b, EGE 5, EGE 15-a, EGE 20, EGE 22, and SC – see explanations for Tables No. 1, 2, and 3; 2. The figure should be read together with Tables No. 1, 2, and 3; 3. The source of this figure is paper [7]**

**Main material and results.**

We consider that one of the problems is that, prior to tunnel construction, the foundation was strengthened by means of a jet grouting technology [5, 6].

It should be emphasized that current Ukrainian construction standards have no recommendations regarding the foundation strengthening by jet grouting for the subsequent construction of underground excavations in them [8, 9].

The engineering-geological profile of the inclined tunnel within the accident site is shown in Fig. 1.

The figure shows that foundation strengthening by means of jet grouting within the accident site was carried out in gravelly soil with clay interlayers and chaotic inclusions of large fragments of weathered rock and concretions (IGE 15-a).

We suggest that in this case the technological problems were as follows:

1. Adequate erosion and uniform mixing when transitioning from a clay layer to a gravel soil layer (and vice versa) is impossible without changes in the technological parameters of the process.

2. If a large fragment is encountered along the path of monitor movement, it is impossible to get the SC design dimensions (i.e. the SC length is reduced).

From the above, the following can be concluded:

1. It is quite probable that soil-cement columns will be either of a smaller or of a larger diameter (compared to the design values); the soil cement will have inclusions of clay blocks; and, as a result, the columns with a heterogeneous structure and reduced strength at their periphery will be formed.

2. It is quite probable that the SC length will decrease relative to its design value.

Since scientific and technical literature contains practically no data on the calculation of the SC strength and stability above underground excavations, consider this problem in more detail.

We will calculate the strength and stability of a soil-cement column (SC) assuming that, due to the characteristics of the soil in which it is constructed, there is a certain deviation of its diameter from the design value (Fig. 2).

The problem was formulated as follows:

1. Within the range of the considered depths, the mass structure of soil is known as well as its normative and design characteristics.

2. The diameter and the normative and design characteristics of the soil-cement element (SC) are available.

3. The Coulomb – Mohr law is adopted as the criterion for the soil and soil-cement strength.

4. To assess the stability of a soil-cement element, a stability coefficient $K_{st}$ was introduced, numerically equal to the ratio of the resultant of the forces retaining a soil-cement element $T_{ud}$ to the resultant of shearing forces $T_{sd}$:

$$K_{st} = \frac{T_{ud}}{T_{sd}}, \qquad (1)$$

In this case, if $K_{st} > 1$, then a soil-cement element is in a stable state; if $K_{st} = 1$, then a soil-cement element is in a neutral state; and if $K_{st} < 1$, then a soil-cement element is in an unstable state (i.e. failure of the soil mass above the excavation will take place).

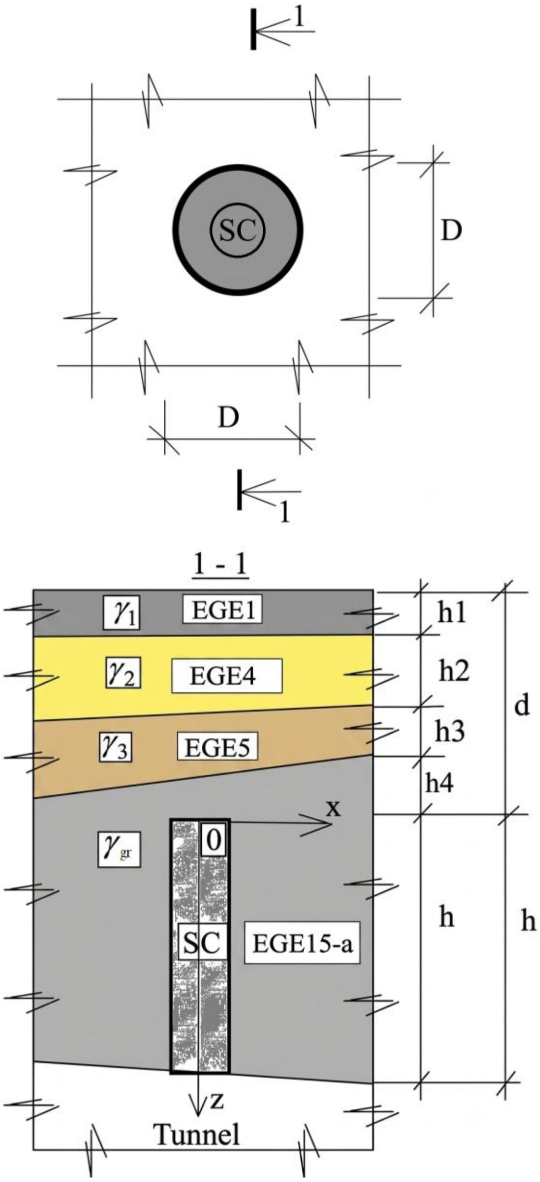

**Figure 2 – Scheme for calculating the stability of a soil-cement element (SC). Note: the figure should be considered together with Fig.1 and Table 1.**

Next, consider the forces acting on a soil-cement element.

1. The SC top is affected by the pressure from the self-weight of soil $P$, which is numerically equal to:

$$P = \gamma_{avg} \cdot d, \qquad (2)$$

where $\gamma_{avg}$ is weighted average value of the specific weight of the soil within the depth interval from the daylight surface to the SC top; and $d$ is distance from the daylight surface to the SC top. Here:

$$\gamma_{avg} = \frac{\sum_{i=1}^{n} \gamma_i \cdot h_i}{\sum_{i=1}^{n} h_i}, \qquad (3)$$

where $d = \sum_{i=1}^{n} h_i$; $\gamma_i$, and $h_i$ are specific weight and thickness of the $i^{th}$ soil layer, respectively; and $n$ is number of soil layers within the interval of depths from the SC top to the daylight surface.

2. The SC weight is equals to

$$G_{gce} = \gamma_{gc} \cdot \pi \cdot \frac{D^2}{4} \cdot h, \qquad (4)$$

where $\gamma_{gc}$, $D$, and $h$ are specific weight, diameter, and height of a SC, respectively.

3. Taking into account points 2 and 3, shearing force is equal to:

$$T_{sd} = \pi \cdot \frac{D^2}{4} \cdot \left(P + \gamma_{gc} \cdot h\right), \qquad (5)$$

4. Vertical stresses $\sigma_z$ acting within the depth interval $z \in (-d, h)$ are equal to (see Fig. 2):

$$\sigma_z = P + \gamma_{gr} \cdot z. \qquad (6)$$

Here $\gamma_{gr}$ is specific weight of soil along the SC lateral surface, and $z$ is design depth (Fig.2)

5. The horizontal pressure from soil $\sigma_x$ acting on a SC is equal to:

$$\sigma_x = \xi \cdot \sigma_z = \xi \cdot \left(P + \gamma_{gr} \cdot z\right), \qquad (7)$$

where $\xi_{gr} = \frac{\nu_{gr}}{1 - \nu_{gr}}$ is coefficient of lateral pressure from the soil, and $\nu_{gr}$ is Poisson's ratio of the soil where a SC is located.

6. Tangential stress of the retaining forces at the design depth $z$ within the framework of the Coulomb–Mohr strength theory is equal to:

$$\begin{aligned} \tau_{ud} &= \sigma_x \cdot tg\left(\varphi_{gr}\right) + c_{gr} = \\ &= \xi \cdot \left(P + \gamma_{gr} \cdot z\right) \cdot tg\left(\varphi_{gr}\right) + c_{gr} \end{aligned}, \qquad (8)$$

where $\varphi_{gr}$ and $c_{gr}$ are angle of internal friction and specific cohesion of the soil adjacent to the SC lateral surface, respectively.

7. The retaining force acting on SC, taking into account (8), is found as:

$$T_{ud}=\pi\cdot D\cdot\int_{0}^{h}\tau_{ud}\cdot dx=$$

$$=2\cdot\pi\cdot D\cdot h\cdot\left[\xi_{gr}\cdot\left(\frac{\gamma_{gr}\cdot h}{2}+P\right)\cdot tg\left(\varphi_{gr}\right)+c_{gr}\right] \qquad (9)$$

8. Taking into account (5), (8), and (9), the SC stability coefficient with respect to the soil is equal to:

$$k_{st}=\frac{8\cdot h\cdot\left[\xi_{gr}\cdot\left(\frac{\gamma_{gr}\cdot h}{2}+P\right)\cdot tg\left(\varphi_{gr}\right)+c_{gr}\right]}{D\cdot\left(P+\gamma_{gc}\cdot h\right)} \qquad (10)$$

Next, analyze (10).

1. First, assume that the SC top reaches the daylight surface. In this case, in (10) we set P=0. We obtain:

$$k_{st,0}=\frac{4\cdot\left[\xi_{gr}\cdot tg\left(\varphi_{gr}\right)\cdot\gamma_{gr}\cdot h+2\cdot c_{gr}\right]}{D\cdot\gamma_{gc}} \qquad (11)$$

2. Next, assume that the SC top is at an infinite depth. In this case, in (10) we set P=∞. We obtain:

$$k_{st,\infty}=\frac{8\cdot tg\left(\varphi_{gr}\right)\cdot h}{D}. \qquad (12)$$

3. Further, we determine the ratio of the stability coefficients for a SC located at an infinite and zero ,

$$\eta=\frac{k_{st,\infty}}{k_{st,0}}=\frac{2\cdot\gamma_{gc}\cdot tg\left(\varphi_{gr}\right)\cdot h}{\xi_{gr}\cdot tg\left(\varphi_{gr}\right)\cdot\gamma_{gr}\cdot h+2\cdot c_{gr}} \qquad (13)$$

4. Next, we determine the ratio of the stability coefficients for a SC located at an infinite and zero depth $\eta=\frac{k_{st,\infty}}{k_{st,0}}$ for an ideally cohesionless soil (to do this, it is necessary to set $c_{gr}$ = 0). We obtain:

$$\eta=\frac{k_{st,\infty}}{k_{st,0}}=\frac{2\cdot\gamma_{gc}}{\xi_{gr}\cdot\gamma_{gr}}. \qquad (14)$$

From (14), it follows that the stability coefficient for an SC constructed at an infinite depth is more than twice as high as that for an SC whose head is located at the daylight surface. In particular, this fact confirms the widely accepted scientific view that the greater the depth of excavation construction, the lower the probability of an accident during its construction [10, 11].

At the same time, from (12) it follows that at an infinite depth, the SC stability coefficient practically does not depend either on the properties of the soil above its head or on the specific weight of the soil-cement.

Next, using our theoretical results (formula (10)), verify the assumption of the authors of [1] regarding how possible deviations from the SC design dimensions and the technology of excavation works in the inclined shaft applying a drilling-and-blasting method could have affected the strength and stability of its roof.

**Table 1 – Design properties of the foundation-forming soils according to the first and second groups of limit states**

| No. | Number of an engineering-geological element | Soil and soil-cement properties | | | | | |
|---|---|---|---|---|---|---|---|
| | | Specific weight of soil, kN/m³ | | Angle of internal friction, kPa | | Specific cohesion, kPa \| | |
| | | $\gamma_{II}$ | $\gamma_{I}$ | $\varphi_{II}$ | $\varphi_{I}$ | $c_{II}$ | $c_{I}$ |
| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
| 1 | EGE 1 | 16.0 | 15.6/16.4 | - | - | - | - |
| 2 | EGE 4 | 18.1 | 16.8/19.4 | 23 | 22/24 | 15.0 | 10.0/20 |
| 3 | EGE 5 | 16.8 | 16.0/17.6 | 31 | 30/32 | 2.0 | 0.8/3.2 |
| | EGE 5,sat | 19.3 | 18.4/20.2 | 31 | 30/32 | 2.0 | 0.8/3.2 |
| 4 | EGE 15-a | 20.8 | 1.98/21.8 | 40 | 37/43 | 1.0 | 0.7/3.2 |
| 5 | EGE 20 | 20.3 | 1.93/21.3 | 35 | 32/38 | 2.0 | 0.7/1.3 |
| 6 | EGE 21 | 19.6 | 18.7/20.5 | 23 | 22/24 | 17.0 | 11.0/23 |
| 7 | EGE 22 | 22.3 | 21.2/23.4 | 34 | 31/35 | 45.0 | 30.0/60 |

Notes: 1. Following designations are used in Table 1: EGE 1 is fill soil (asphalt, concrete, black loam, and yellow-brown loam); EGE 4-b is sandy loams, loams, yellow clays, yellow-brown, grey, and greenish-grey soils with lenses of silty sand; EGE 5 is fine quartz sands, grey, light grey, light yellow, dense, with lenses of poorly graded sand and sandy loam; EGE 15-a is gravelly strata (poorly graded sands with inclusions of pebbles and gravel of crystalline rocks, with a content of 20-40%, with interlayers of clay material); EGE 20 is clays (glauconitic rocks of greenish-grey colour with varying degrees of alteration, semi-hard, with lenses and interlayers of quartz-glauconite sand and sandy loam); EGE 21 is dispersed zone of the weathering crust of rock formations – primary kaolin with inclusions of crushed stone and rock debris, stiffplastic; and EGE 22 is fragmental zone of the weathering crust of rock formations with clayey filling.

2. In Table 1, the lower indices “II” correspond to soil characteristics calculated for a confidence probability interval of 0.85 (second group of limit states), while the lower indices “I” correspond to soil characteristics calculated for a confidence probability interval of 0.95 (first group of limit states).

3. In Table 1, for the design soil characteristics $\gamma_I$, $\varphi_I$, and $C_I$, the numerator shows their minimum values (i.e. $\gamma_{I,\min}$, $\varphi_{I,\min}$, and $C_{I,\max}$), while the denominator shows the maximum values (i.e. $\gamma_{I,\max}$, $\varphi_{I,\max}$, and $C_{I,\max}$).

4. This table should be read together with Figure 1.

5. The source for this table is paper [7].

First, determine the length of a SC at which its bearing capacity with respect to the soil is lost. Assume that, due to the heterogeneity of the soil mass, there is a deviation from the SC design diameter (800 mm) by 200 mm, being either smaller or larger. Therefore, we consider three cases of the soil-cement column diameters:

1. The SC diameter corresponds to its design value (D=800 mm).

2. The SC diameter is 200 mm larger than the design value (D = 1000 mm).

3. The SC diameter is 200 mm smaller than the design value (D = 600 mm).

At the same time, we assume the stability coefficient to be equal to a unit (this case corresponds to a neutral state of a SC).

Since the problem of SC strength and stability with respect to the soil is being solved, in the numerator of (10) it is necessary to set the minimum values of the calculated soil characteristics determined within the confidence probability interval $\alpha$=0.95 (i.e. $\gamma_{I,\min}$, $\varphi_{I,\min}$, and $C_{I,\min}$), and in the denominator, their maximum values should be set (i.e. $\gamma_{I,\max}$, $\varphi_{I,\max}$, and $C_{I,\max}$).

In this case, we obtain the most unfavourable state, as required by Ukrainian construction standards [12, 13]. Taking into account the above, formula (10) changes as follows:

$$k_{st} = \frac{8 \cdot h \cdot \left[ \xi_{gr} \cdot \left( \frac{\gamma_{grI,\min} \cdot h}{2} + P_{I,\min} \right) \cdot tg\left(\varphi_{grI,\min}\right) + c_{grI,\min} \right]}{D \cdot \left( P_{I,\max} + \gamma_{gcI,\max} \cdot h \right)}, \tag{15}$$

where:

$$\left. \begin{aligned} P_{I,\min} &= \frac{\gamma_{1,I,\min} \cdot h_1 + \gamma_{2,I,\min} \cdot h_2 + \gamma_{3,I,\min} \cdot h_3 + \gamma_{grI,\min} \cdot h_4}{d}; \\ d &= h_1 + h_2 + h_3 + h_4. \end{aligned} \right\}, \tag{16}$$

and

$$\left. \begin{aligned} P_{I,\max} &= \frac{\gamma_{1,I,\max} \cdot h_1 + \gamma_{2,I,\max} \cdot h_2 + \gamma_{3,I,\max} \cdot h_3 + \gamma_{grI,\max} \cdot h_4}{d}; \\ d &= h_1 + h_2 + h_3 + h_4. \end{aligned} \right\}, \tag{17}$$

Here: $\gamma_{1,I,\min}, \gamma_{2,I,\min}, \gamma_{3,I,\min}$, and $\gamma_{grI,\min}$ are minimum values of the soil specific weights calculated within the confidence probability interval $\alpha$=0.95 (see Table); $\gamma_{1,I,\max}, \gamma_{2,I,\max}, \gamma_{3,I,\max}$, and $\gamma_{grI,\max}$ are maximum values of the soil specific weights calculated within the confidence probability interval $\alpha$=0.95 (see Table 1); and $h_1, h_2, h_3$, and $h_4$ are thicknesses of the soil layers along the design vertical above the SC top.

Figure 3 presents the dependencies of the stability coefficient of a SC on its length before the groundwater level rises. The SC diameters of 600, 800, and 1000 mm are considered (the design diameter is 800 mm). Such a range was adopted because, due to the heterogeneity of the foundation and impossibility of adjusting process parameters, the SC diameter deviation from the design value may reach 20-30%.

Similar dependencies, assuming that the groundwater is at a depth of 1.0 m from the daylight surface, are shown in Figure 4.

The analysis of the data presented in the figures and their comparison allowed us to draw the following conclusions:

1. If the influence of groundwater is not taken into account, the calculated stability coefficient of a SC varies from 0 to 11. At the same time, the larger the SC diameter, the lower the stability coefficient.

2. When calculated taking into consideration the buoyancy effect of groundwater, the stability coefficient varies from 0 to 6.2. At the same time, the larger the SC diameter, the lower the stability coefficient.

3. Without accounting for the influence of groundwater, a stability coefficient equal to unit is reached at the following SC lengths:

- 306 mm for a diameter of 600 mm;

- 412 mm for a diameter of 800 mm; and
- 520 mm for a diameter of 1000 mm.

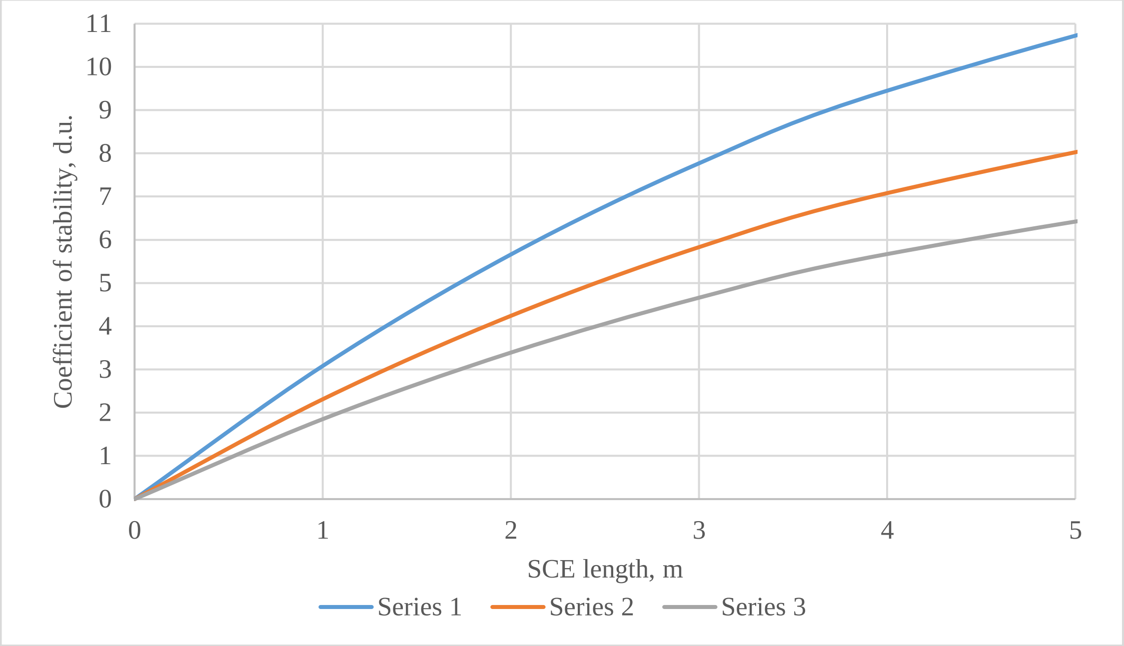

**Figure 3 – Dependence of the stability coefficient of a SC on its length without accounting for the influence of groundwater. Series 1 – SC diameter is 600 mm; Series 2 – SC diameter is 800 mm; Series 3 – SC diameter is 1000 mm**

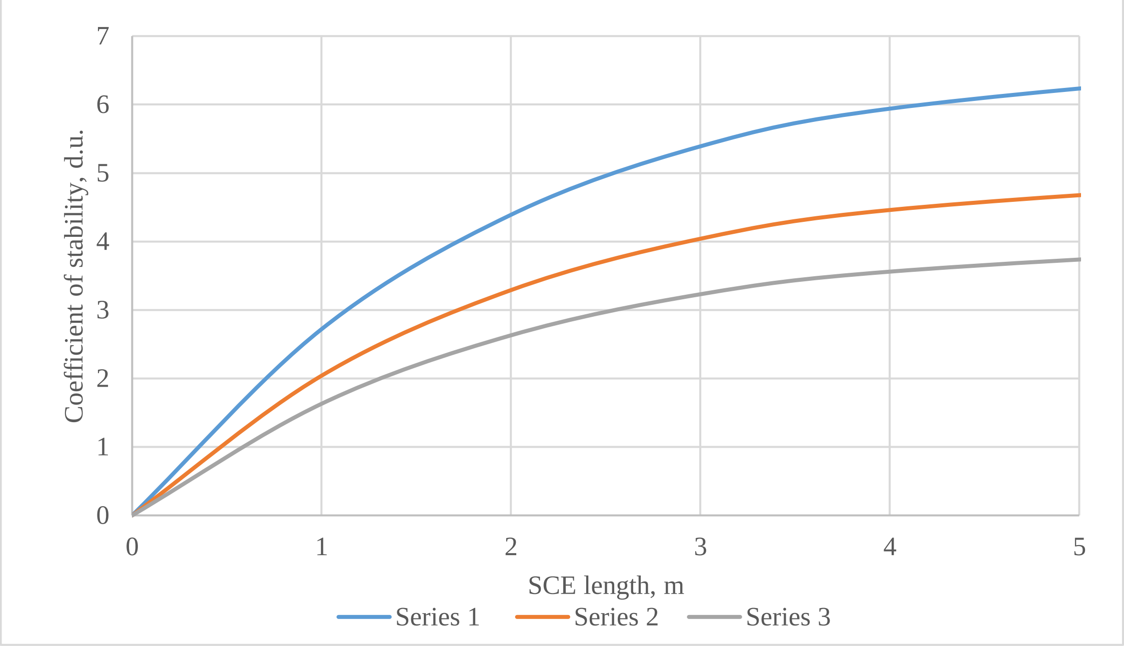

**Figure 4 – Dependence of the stability coefficient of a SC on its length after the groundwater level rises**

4. Taking into account the buoyancy effect of groundwater, a stability coefficient equal to unity is reached at the following SC lengths:

- 314 mm for a diameter of 600 mm;
- 430 mm for a diameter of 800 mm; and
- 534 mm for a diameter of 1000 mm.

It is also of interest to use formula (15) to assess the stability of a group of soil-cement columns. For this purpose, in (15) diameter D should be replaced by so-called equivalent diameter D’, which should be determined using the formula:

$$D^{'} = \sqrt{F_{pr}}\,, \qquad (18)$$

where $D^{'}$ is equivalent diameter and $F_{pr}$ is cross-sectional area of a group of soil-cement columns.

The initial data for the calculation, using formulas (14) and (18), were taken from paper [1] (i.e. the statement: "…it was recorded that on the surface around the projection of the upward shaft face, within the contours of jet strengthening of rock using soil-cement columns by a jet grouting technology, five conical sinkholes with an area of 5.0…51.0 m² and a depth of 0.8…6.0 m were formed…").

For a sinkhole with an area of 5.0 m², we obtain: $D' = \sqrt{F_{pr}} = \sqrt{5} = 2.23$ m, from which, taking into account (15) at $k_{st} = 1$, the height of the SC group that lost its stability (and consequently the sinkhole depth) is $h$ = 1.2 m, which corresponds to the field measurement data shown in [1].

For a sinkhole with an area of 51.0 m², we obtain: $D' = \sqrt{F_{pr}} = \sqrt{51} = 7.14$ m, from which, taking into account (15) at $k_{st} = 1$, the height of the SC group that has lost its stability (and consequently the depth of a sinkhole formed as a result of SC failure) is $h = 6.1$ m, which corresponds to the field measurement data presented in [1].

A general conclusion was made that one of the possible causes of accidents could be either a SC deviation from the design dimensions (i.e. a reduced SC length along with its changed diameter) or a complete absence of SCs within the tunnel excavation area.

The next stage of the study included investigation of the process of changes in the SC stability coefficient with a decreasing internal friction angle of the soil.

The necessity of carrying out these studies is stipulated by the fact that during the excavation of underground workings by a drilling-and-blasting method and, as a consequence, under dynamic loading of the soil (especially water-saturated one), its strength decreases sharply, up to the transition into a free-flowing state [14, 15, 16].

During the numerical experiment with the use of formula (10), we constructed graphical dependencies of the length of a soil-cement column on the angle of internal friction along its lateral surface at a stability coefficient equal to unit (this case corresponds to the initial stage of soil failure along the SC lateral surface).

The obtained curves for the unsaturated foundation (i.e. without accounting for the buoyancy force) are shown in Figure 5; the curves for the water-saturated foundation are shown in Figure 6.

It was found that failure of the unsaturated foundation at the design SC length of 5 m takes place when the internal friction angle decreases to 2-5 degrees, whereas failure of the water-saturated foundation is observed when the internal friction angle decreases to 5-9 degrees. Thus, water saturation of the foundation significantly reduces the strength of the foundation strengthened by jet grouting

**Key findings:**

1. It was defined that the stability and strength of the soil above excavations significantly depend on deviations in the dimensions of the soil-cement elements from their design values.

2. It was determined that a reduction in strength along the SC lateral surface (a phenomenon that takes place under dynamic loading of the foundation) has a strong effect on the strength and stability of the SCs located in the roof of underground excavations. This confirms the assumption formulated by the authors of [1] that one of the possible causes of the accident during metro construction in the city of Dnipro could have been the method of underground excavation involving drilling-and-blasting operations.

**Conclusion**.

The technology of strengthening the fragmented-soil foundations by jet grouting while driving underground excavations by a mining method is novel and requires detailed study. Therefore, we consider that it is necessary to supplement traditional technologies, as well as methods for the design and calculation of underground workings, with a number of new provisions.

**Recommendations**.

Attention should be paid to following parameters of underground excavation technologies and strengthening the foundation with the help of SCs:

1.1. The method of excavating underground workings in sandy and gravel soils (in this case, dynamic impacts on the soil should be minimized).

1.2. Adjustment of the soil erosion and mixing processes during the formation of the same soil-cement column in different soil layers (e.g. when transitioning from clay layers to sandy, gravel, and coarse-grained soils and vice versa).

1.3. Crushing of large rock fragments and boulders of hard rock encountered while constructing soil-cement columns (these inclusions prevent achieving the SC design length).

1.4. Accurate provision of the SC design dimensions.

1.5. It is advisable to strengthen foundation in such a way that the soil-cement columns intersect in plan by approximately 1/3 of their radius (exact value is 0.273 of the radius, Fig. 7). In this case, a continuous soil-cement monolith is formed, and possible isolation of soil-cement columns is eliminated.

2. When calculating and designing underground excavations in foundations strengthened by jet grouting, the SC stability coefficient (formula 10) should be determined. It should be used as the basis for the following design parameters:

2.1. Depth of the SC top, as well as SC diameter and length.

2.2. Depth of the tops of a group of SCs, equivalent diameter of the SC group, and length of the SC group

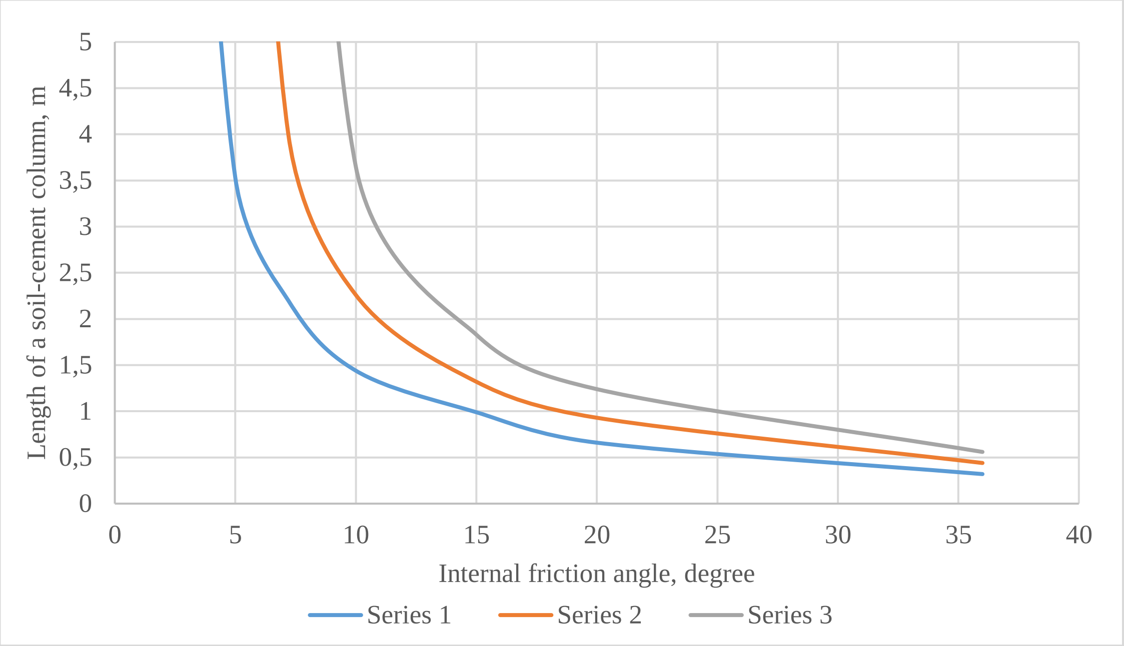

**Figure 5 – Dependence of the SC length on the internal friction angle of the soil at a stability coefficient equal to unit. Note: the calculation did not take into account the buoyancy effect of groundwater.**

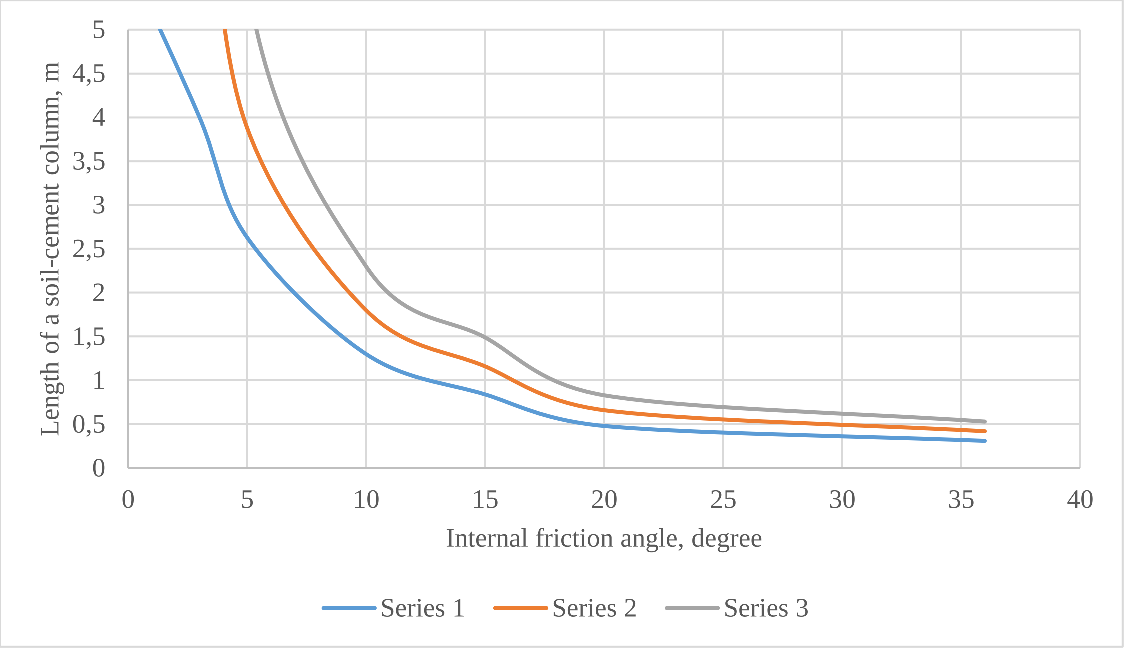

**Figure 6 – Dependence of the SC length on the internal friction angle of the soil at a stability coefficient equal to unit. Note: the calculation took into account the buoyancy effect of groundwater**

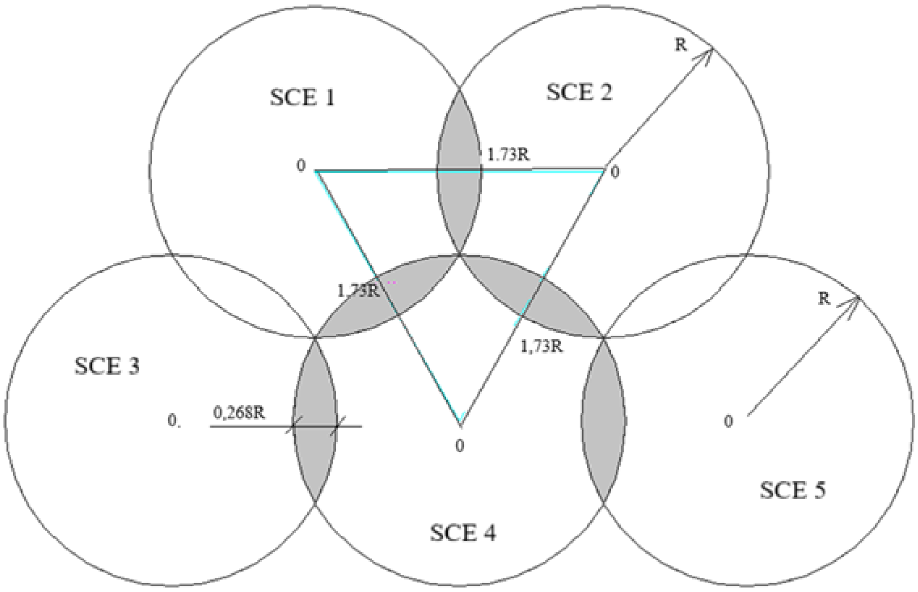

**Figure 7 – Recommended arrangement of soil-cement columns in plan. Notes. Following designations are used in Figure 7: - SC1, SC2, …, SC5 are numbers of soil-cement elements; - R is radius of a soil-cement element.**

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# Локальна стійкість грунтоцементних стовпів над підземною виробкою

Аннотація. Стаття присвячена дослідженню локальної стійкості ґрунтоцементних елементів (ҐЦ), сформованих за струминною технологією (jet grouting), у зоні впливу підземних виробок. Актуальність роботи зумовлена аварією під час будівництва метрополітену в м. Дніпро у 2021 р. та необхідністю оцінки ризиків при проходці виробок у зміцнених основах. Метою дослідження є встановлення впливу геометричних параметрів ґрунтоцементних елементів та зниження їх міцності на стійкість масиву над виробкою. Проаналізовано умови розвитку аварії, зокрема зміну гідрогеологічного режиму, динамічні навантаження від буро-вибухових робіт і порушення структури ґрунтів. Встановлено, що відхилення розмірів ґрунтоцементних елементів від проєктних значень і зниження міцності по їх боковій поверхні суттєво зменшують стійкість покрівлі виробки. Підтверджено значний вплив динамічних навантажень на втрату стійкості. Також сформульовано практичні рекомендації щодо підвищення надійності геотехнічних систем і зниження ризиків аварій.

Ключові слова: ґрунтоцементні елементи, ін'єктування, підземні виробки, динамічне навантаження, гідрогеологічні умови, геотехнічний ризик

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